ABN 81 837 469 730

Report to: Grantham Commission of Inquiry- Investigation of timber utility pole failure

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1.0 Executive Summary

1.1 I was engaged by the Grantham Floods Commission of Inquiry to provide an expert opinion regarding the damage to a timber utility power pole (pole 182127) (the **Failed Pole**). The Failed Pole formed part of a line of power poles at Grantham on Lot 103 on CH 31505.

1.2 My opinion was sought as to the likely cause or causes of failure of the Failed Pole. In providing my opinion I was requested to:

a. Consider the type of timber, probable design and construction and likely condition of the Failed Pole prior to the January 2011 flood;

b. Consider the probable properties of the Failed Pole prior to the flood including, the ultimate (failure) bending moment strength or equivalent yield stress;

c. Determine the estimated horizontal force that would be required to break the Failed Pole (as observed);

d. Determine, to the extent it is possible to do so, the extent to which any (or all) of the following contributed to the failure of the Failed Pole:

   i. the likely direction of water flow;

   ii. the likely velocity of water flow;

   iii. the level of the broken pole above surface level;

   iv. the heights of water flow; and

   v. the floating debris that may have been carried on the water flow.

   vi. an adjacent embankment to the SE, close to the failed pole.

1.3 My investigation included a site inspection, measurements undertaken on site relating to the failed pole and assessments of the condition of the pole and likely causes for the failure.

1.4 Information obtained from the site inspection was supplemented with additional information provided by Energex (Ref 12.1) and by the Commissions Letters of Instructions dated 16 July 2015 and 27 July 2015 respectively (Appendix 5).

1.5 A summary of the findings from my investigation are:

a. The Failed Pole (Pole 182127) failed at greater than 4800mm (vertical projection of bottom of pole fracture from current leaning position) above the current ground level. This would be at approximately 127.85mAHD to 128.55mAHD to the bottom of the fracture if the pole were standing approximately vertically as originally installed.

b. The Failed Pole was blackbutt.

c. The direction of lean on the Failed Pole and also the orientation of the fracture roughly align with the resolved attached conductor (overhead power lines) directions.

d. The bending moment (for a simple cantilevered pole, the load applied at a point multiplied by the distance to the failure point) induced in the pole at the failure was in a clockwise direction when viewed from the South West (SW) and also aligns with the lean of the pole direction.

e. For a load applied near the top of the pole, the dynamic or static stress analysis indicates that the maximum stresses in the pole align closely with the fracture location.

f. The ultimate limit state stress of the pole at the failure location was approximately 49 to 57 MPa.

g. For a simple cantilevered pole (un-stayed), a point load of approximately 31kN (3.1 tonne force) or greater, applied at or near the pole top (X-arms) would be required to cause pole failure.

h. An increase in the height of the ground level at the base of the pole from time of installation to at time of flood event of 200 to 300 mm (or to a height up to the fracture location) would not have had any significant impact on the above estimates as the maximum stresses induced in the pole are well above the ground line.

i. From the evidence and measurements obtained together with assessments and analysis, it could not be concluded if there has been any change in the level of ground surrounding the pole from time of initial pole installation to time of flood event, or thereafter.

j. I consider the most likely scenario for failure of the Failed Pole (Pole 182127) to be flood debris impact with the conductors (power lines) between Pole 182127 and Pole 182128 and/or directly on Pole 182128 and, a less likely scenario being, large debris striking Pole 182127 directly above the break. That is for the most likely scenario, for a simple cantilevered pole fixed (embedded) at its base by the ground, a force (load) transferred to the pole from the conductors (power lines) at the location of the cross-arms, pulling the pole to the SE.

2.0 Introduction

2.1 I have been engaged by the Grantham Commission of Inquiry to provide an expert opinion as to the likely causes of failure of a timber pole. Refer to Appendix 5, Letters of Instructions for the scope of this engagement.

2.2 In order for me to provide my expert opinion, I have:

a. Undertaken a review of the information provided by Energex (Ref 12.1)
b. Undertaken a site inspection and investigation.
c. Undertaken a literature review of the properties of the timber species of the pole.
d. Sought additional advice and expert input from other parties as referenced in this report.
e. Carried out calculations as appropriate to the scope of my engagement.

3.0 Qualifications

3.1 I was educated in Melbourne and graduated from Caulfield Institute of Technology in engineering in 1974 with a Diploma of Engineering (Civil). I have over 35 years' experience specialising in timber engineering and timber technology having worked for CSIRO Division of Forest Products and the Timber Research and Development Advisory Council of Queensland (now Timber Queensland), and more recently as Principal of MacKenzie Consulting. I am a Fellow of the Institute of Engineers, Australia and a Registered Professional Engineer in Queensland. A more detailed outline of my Curriculum Vitae is provided in Appendix 6.

3.2 In preparing my report I had the support of Mr Lex Somerville and Dr Geoffrey Boughton.

3.3 The qualifications and experience of Mr Lex Somerville, BMCC Services, who assisted me with the site inspection and measurements on site are also provided in Appendix 6.

3.4 The qualifications and experience of Dr Geoffrey Boughton, TimberEd Services Pty Ltd, who provided me with pole properties data and information as well as structural analysis input are also provided in Appendix 6.

4.0 Letters of Instructions

4.1 In this report, I have endeavored to address the items detailed in the Letters of Instructions I have received from the Commission. The Letters of Instructions detailing these requirements are included in Appendix 5.

5.0 Site Inspection

5.1 General

5.1.1 Prior to the site inspection, additional information I received included a Statement by Ron Barbagallo, Energex (Ref. 12.1) that provided in part, Failed Pole location information and details about the Failed Pole and associated attachments original installation.

5.1.2 From Ref 12.1, and the aerial photograph of the site given in Figure 1 attached to the , Letter of Instructions (Ref Appendix 5), the Failed Pole, is identified as Pole 182127, and is located on Lot 103 of CH 31505, approximately 500m west of Dorrs Road Grantham. Refer Ref. 12.1, Attachment RAB – 2 for details. For ease of reference, a copy of the Energex site location RAB2 is re-produced in Appendix 1. RAB2 also shows the location of poles 182128 and 182126.

5.1.3 Mr Lex Somerville - BMCC Services and I, undertook the site inspection and assessment on 26 June 2015. Also present during the inspection were:

a. Mr Arthur Simpson – Safety Management Systems Manager, Energex and

b. Mr Andrew East – Boral

5.2 WH&S

5.2.1 Prior to commencement of the on-site inspections, a Job Safety and Environmental Impact Analysis (JSA), previously prepared by me, was reviewed, amended and signed off by all parties present.

5.2.2 A requirement by Energex prohibited the leaning of a ladder against the pole to be inspected and this limited the ability to inspect the fracture point in close detail. Mr Simpson also provided a non-conductive extendable measuring rod which assisted with the site measurements.

5.3 Site Observations (General)

5.3.1 I identified the Failed Pole location (Pole 182127) to be consistent with the location identified by Energex. (Ref. 12.1). It is located on an embankment adjacent to a water body (presumably a disused water filled quarry). Also refer to Image 1. below reproduced from Reference 1.

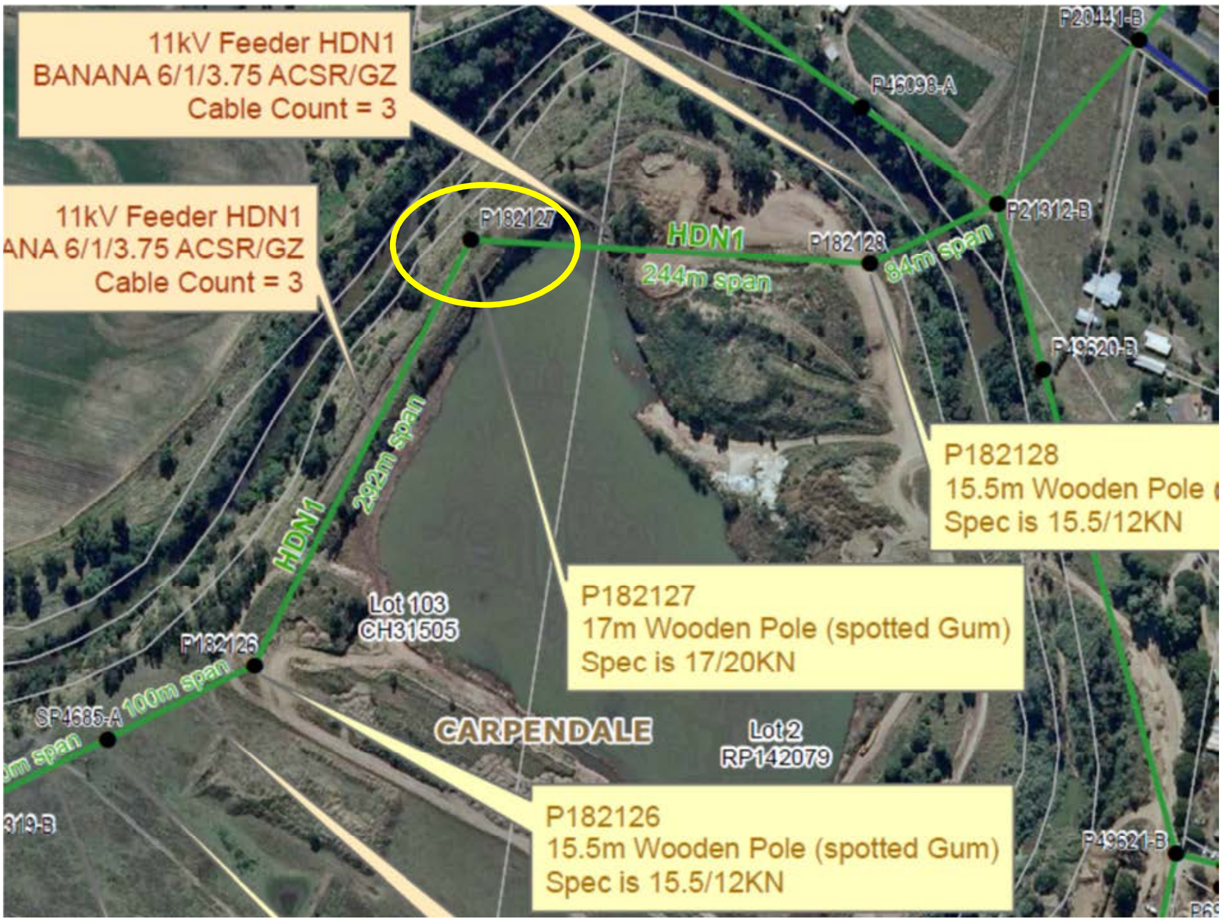


Image 1. – Location of failed pole 182127.

5.3.2 No visible evidence of stays as indicated in Ref. 12.1, RAB 4, or their anchor points for the Failed Pole (Pole 182127) were found by Mr Somerville, Mr East or Mr Simpson who investigated for possible evidence of them in the relatively long grass, on site, during the inspection.

5.3.3 From base to tip, I identified the pole was leaning in an approximate south-east direction towards the water body. Refer to Appendix 3, Photo's 1 to 3 for a general view of the failed pole and the direction of lean.

5.3.4 I found the original pole identification disc, pole number and current leakage test point (coach screw) still to be intact on the remaining section of the pole. Refer to Appendix 3, Photo 4 and 5 that show the disc, identification number and test point.

5.3.5 I identified visible bruising/scarring/damage at a number of locations on the remaining section of the pole. Refer Appendix 3, Photo's 6 and 7 that show these damage points.

5.3.6 I identified some debris (logs, barbed fencing wire and star pickets) wrapped around/attached to the base of the pole and I assume this to have occurred from the flood event under consideration. Refer Appendix 3, Photo 8 that shows this debris.

5.3.7 I noticed what appeared to be the top of an old fence post to be next to the pole on the SE side. Refer Appendix 3, Photo 9 that shows this possible fence post.

5.3.8 I observed a slight colour variation on the surface of the pole for a distance of approximately 1100 mm from ground line. Refer Appendix 3, Photo 10 which has been digitally enhanced to show this slight colour variation. Note: For comparison, a non-enhanced photo of the same section of pole is shown in Photo 6, Appendix 3.

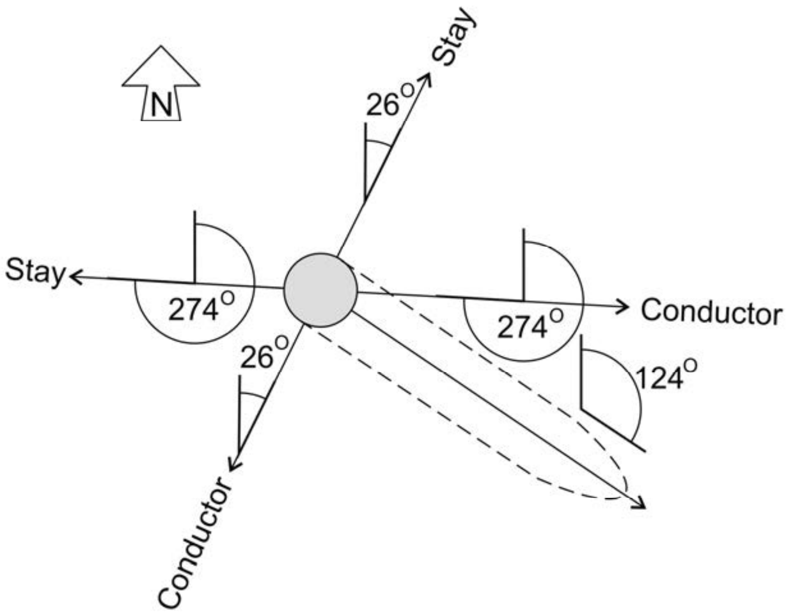
5.3.9 I did not observe any obvious ‘ground line’ deterioration indicators (lines of decay or timber surface degradation that are usually evident with poles embedded for long periods of time) visible on the surface of the pole.

5.3.10 I observed some possible ground line scouring that I noted at the front (NW) and back (SE) side of the post. Refer Appendix 3, Photo’s 11 and 12 that show this possible scouring.

6.0 Site Measurements

6.1 Pole Orientation

By use of an “I phone” compass app, the north point and the direction of lean of the pole were estimated by me. The approximate north point was marked on the pole with a paint dot by Mr Lex Somerville. See Photo 1, Appendix 3 that shows the approximate north point on the leaning pole. I found the pole to be leaning in a SE direction at approximately 124^0 of North. Refer Figure 1. Figure 1 also indicates the approximate orientation of the stays that had been attached to the top of the pole as advised (Ref 12.1.).



Notes:

1. A stay is a diagonal brace (usually a multi strand metal cable) that attaches to just below the cross arms near the top of the pole and runs at an angle to an anchor point at the ground. It provides support to the top of the pole to counter balance forces from wires attached to the top of the pole.
2. Conductors are the overhead power lines.

Figure 1 – Pole 182127 Orientation

6.2 Failed Pole Length and Angle of Lean

Under my direction, the distance from the mean ground line to highest point of the fracture was measured by Mr Somerville and Mr Simpson using the measuring rod provided by Mr Simpson and found to be approximately 5800mm and to the lowest point of the fracture on the opposite side 4800mm. Mr Somerville and I, using a large carpenter’s adjustable protractor and a spirit level found the pole was leaning at approximately 28^0 to the vertical. This was also confirmed independently by an 'I phone' app by Mr East. See Figure 2.

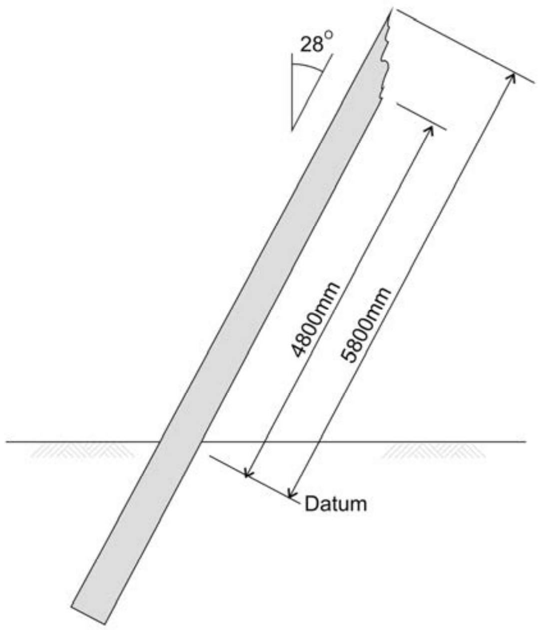
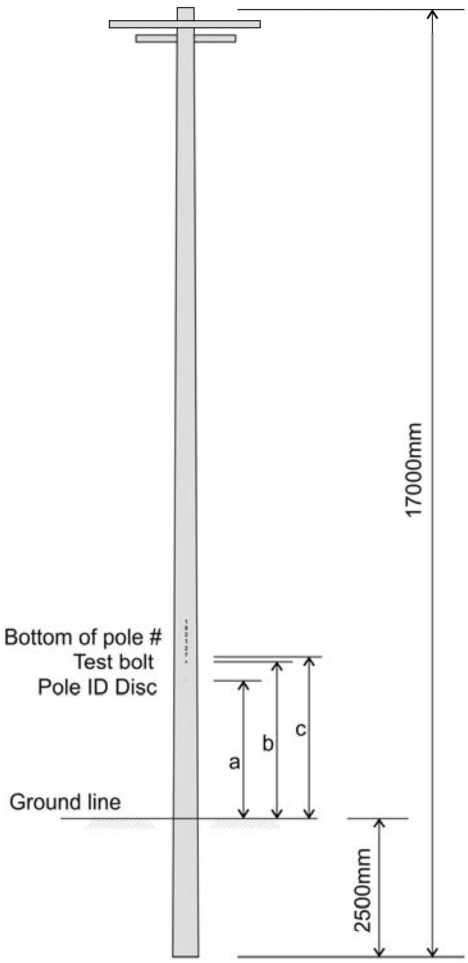


Figure 2 – Pole Length

6.3 Measured Pole Identification Point Locations and Assumed Original Pole Installed Height and Embedment Depth

6.3.1 Figure 3 provides assumed original pole installation height and embedment depth as previously advised by Energex (Ref 12.1) and in the Commissions Instructions (Appendix 5) as well as measured distances from current mean ground level to pole identification disc, pole # and test point. Mr Somerville undertook these measurements on site under my direction using a steel tape.

6.3.2 Refer to Paragraph 6.7 for further information and discussion on the assumed original embedment depth versus that measured on site.



> **NOTE:** a, b and c were measured in-situ on the leaning pole to current mean ground line and do not necessarily relate to the actual dimensions at time of original pole installation.
> a = 2450mm to pole ID disc
> b = 2800mm to test bolt
> c = 2890mm to bottom pole ID #
>
> Original pole height (17000mm) and embedment (2500mm) are based on Energex advice (Ref. 1) together with subsequent Energex advice via Commission.

Figure 3 – Failed Pole (Pole 182127) - Measured ID locations and assumed original pole installed depth and height

6.4 Pole Diameter and estimated Taper

6.4.1 Under my direction, the circumference of the pole was measured at two locations by Mr Somerville using a steel tape. The first being as close to the ground as possible (approximately 400 mm up from mean ground level) and the second as high as could be reached safely from a step ladder being 2920 mm from the first location. Results obtained were 1440mm and 1280mm respectively. Refer to Figure 4.

6.4.2 From this, I estimated the diameters at each location as 458mm and 407mm respectively. The taper in the pole diameter was then calculated from these values by me and found to be approximately 17.4 mm/m length of pole.

6.4.3 Extrapolating these measurements I estimated pole diameters at the highest point of the fracture and lowest point of the fracture of 364 mm and 381 mm respectively and at ground line of 465 mm and an estimated diameter at the pole tip of approximately 220 to 230 mm.

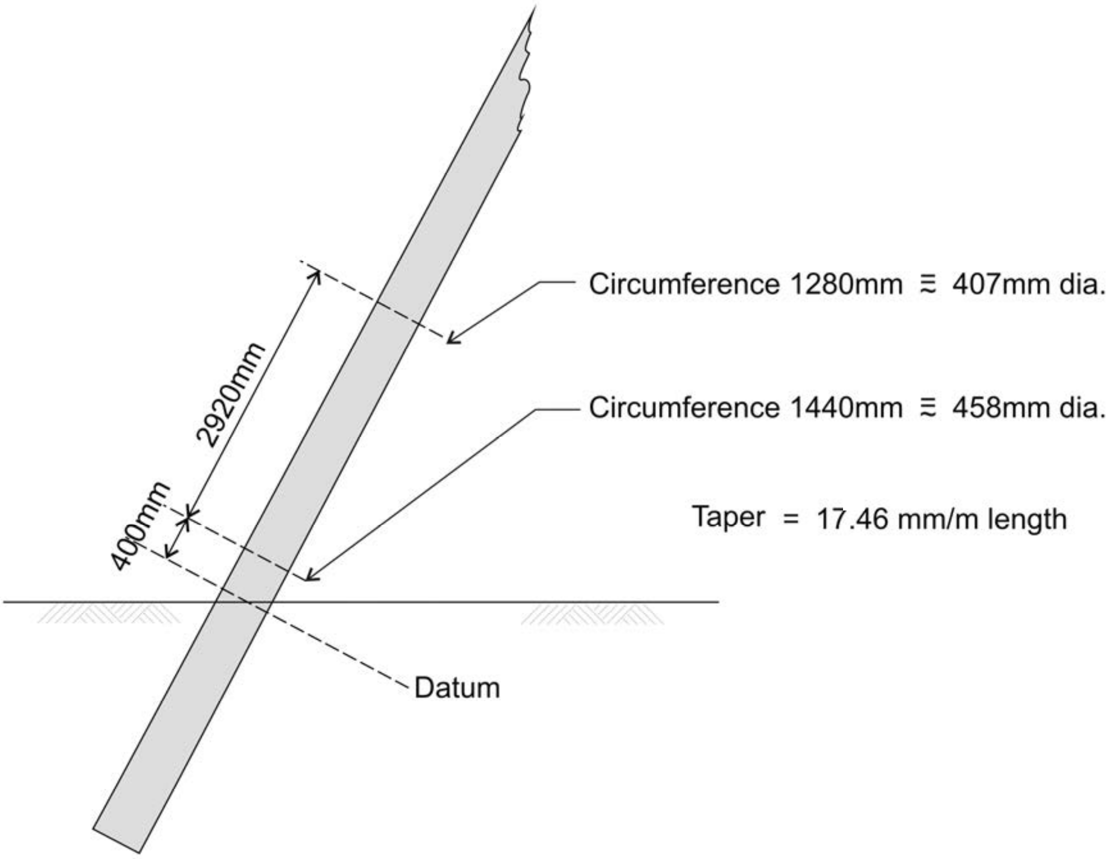


Figure 4 – Pole Circumferences and Taper

6.5 Scouring at Base of Pole

Under my direction, Mr Somerville, using a steel tape measured the depth of scouring to the pole, apparent at the time of inspection. To the SE side of the pole (presumably the downstream side) this was found to be approximately 800 mm from the ground surface on that side of the pole however, there was a lot of debris in the hole so this probably does not represent actual scour depth. Similarly, on the NW (again presumed upstream side of pole) the depth of scouring was found to be approximately 250 mm from the actual ground surface. Refer Appendix 3, Photo's 11 and 12 that show the apparent scouring to the NW side and SE side respectively.

6.6 Damage to Pole Surface

6.6.1 I observed three damage locations on the outer surface of the pole towards the NW face. I did not observe any significant obvious damage on the SE face of the pole. These damage locations are indicated in Figure 5 below and shown in Photographs 13 to 17 in Appendix 3.

6.6.2 These damage locations were located below the pole failure location.

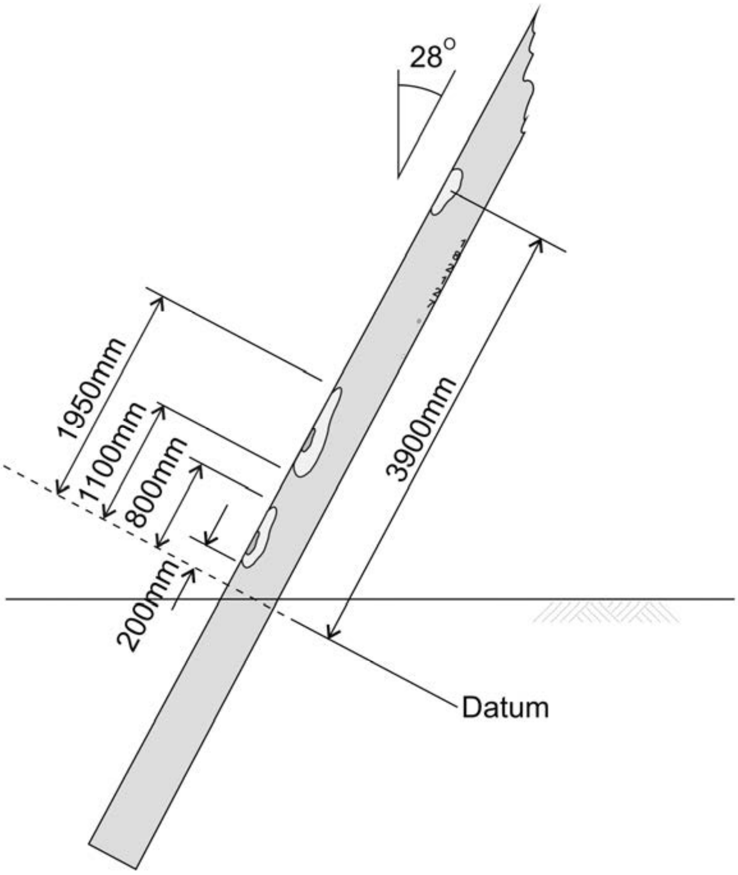


Figure 5 – Location of evidence of damage to pole (Approximate bruising locations indicated by darker shading)

6.6.3 The damage appears to be the result of either/or a combination of abrasion, bruising and shelling. Shelling can occur in timber where there is a line of circumferential weakness that follows the growth rings. It may be due to the presence of gum veins or some other 'weakness' that has arisen such as onset of decay to the inner portion of the sapwood in the pole.

6.6.4 At both the lower damage location and the middle damage location, there is evidence of a bruising depression that is likely to have resulted from some type of impact. This is highlighted in Photo's 14 and 16 in Appendix 3.

6.7 Embedment Depth

6.7.1 The pole disc is located at 5500 mm from the pole butt and typical pole embedment depths (ground line to butt) at time of installation is 10% of pole length plus 800 mm. (Refer Letter of Instructions, Appendix 5). Nominal embedment depth for the pole would therefore be 2500mm. Refer to Paragraph 6.3, Figure 3.

6.7.2 If it is assumed (there are other possibilities – refer Paragraphs 6.7.3 and 6.7.5) the butt of the pole has remained relatively close to the point at which it was originally installed, then I estimate:

a. For an original embedment of 2500 mm, the angled embedment of the leaning pole to ground surface would be approximately 2830 mm (2500 mm/Cos 28 Deg).

b. The measured distance from the pole ID disc to the current mean ground line is 2450 mm. Therefore, if the pole disc was installed at 5500 mm from the butt, then the depth from current mean actual ground level to the butt of the pole should be 3050 mm.

6.7.3 Confirmation that the butt of the pole is still relatively close to the point at which it was first installed would require further geotechnical investigation, such as excavation adjacent to the in-situ pole, down to the butt end of the pole, to enable the estimate above to be further validated.

6.7.4 Based on the above measurements and assumptions, I estimate that it is possible that the current ground line is higher than the assumed ground line at time of pole installation. See Figure 6 for measurements.

6.7.5 If however the butt of the pole has rotated upward from its original installed position, then this would influence the above estimate.

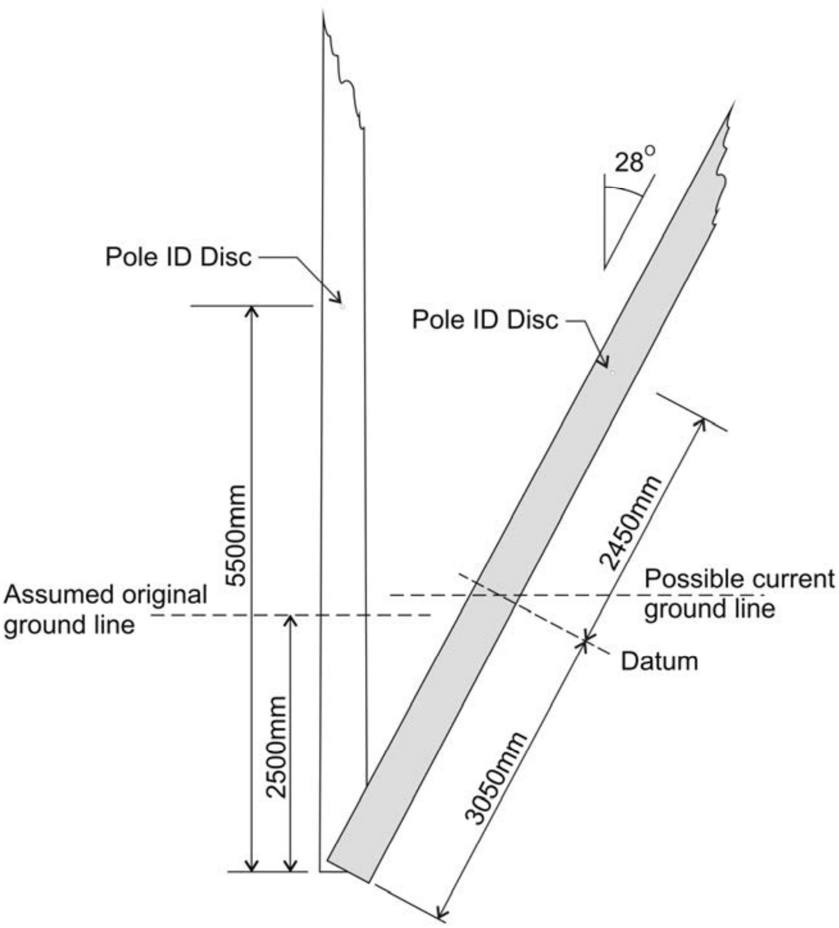


Figure 6 – Measurements of pole embedment versus ground level

6.8 Review of Nature of Pole Fracture

6.8.1 A close examination of the fracture location I conducted (refer Photographs 18 to 21 Appendix 3) indicates:

a. The presence of brittle heart with a brash fracture in the central portion of the pole. Refer Photo 21, Appendix 3 which highlights brittle fracture zone on the compression side of the fracture. Refer also to Paragraph 7.3 which considers the effects of brittle heart in more detail.
b. Elongated fibres bent towards the SE at both top and bottom of the fracture. Refer Photo19, Appendix 3, that highlights these points.
c. That the tension side of the fracture is on the high side (NW) of the fracture and the compression on the low side of the fracture. My reasons for forming this opinion are:
    i. The direction and the angle of broken fibres at both the top and bottom of the fracture. Refer Appendix 3, Photo's 18 and 19 that indicate these directions.
    ii. Signs of compression damage, such as fibres that have buckled, which can be seen in some layers of fibres just in from the face near the lower edge. Refer Appendix 3, Photo 20 that shows this compression damage.
    iii. The small plug close to the neutral axis that has been 'pulled' up as shown in Appendix 3, Photo 19 as highlighted.
    iv. The tension edge moving upward and to the left that has dragged some fibres to the left as shown in Appendix 3, Photo 19.
    v. Rotation to the left about the compression edge at failure which has pulled the fibres at the lower edge to the left. The tension side and compression side are shown in Figure 7 below and in Appendix 3, Photo 19.

6.8.2 I therefore conclude that the moment applied to the pole to induce the fracture was in a clockwise direction when viewed from the SW. See Figure 7. The implications of this are that either:

a. A load was applied above the fracture in a SE direction or
b. A load was applied above and or below the fracture in a NW direction.

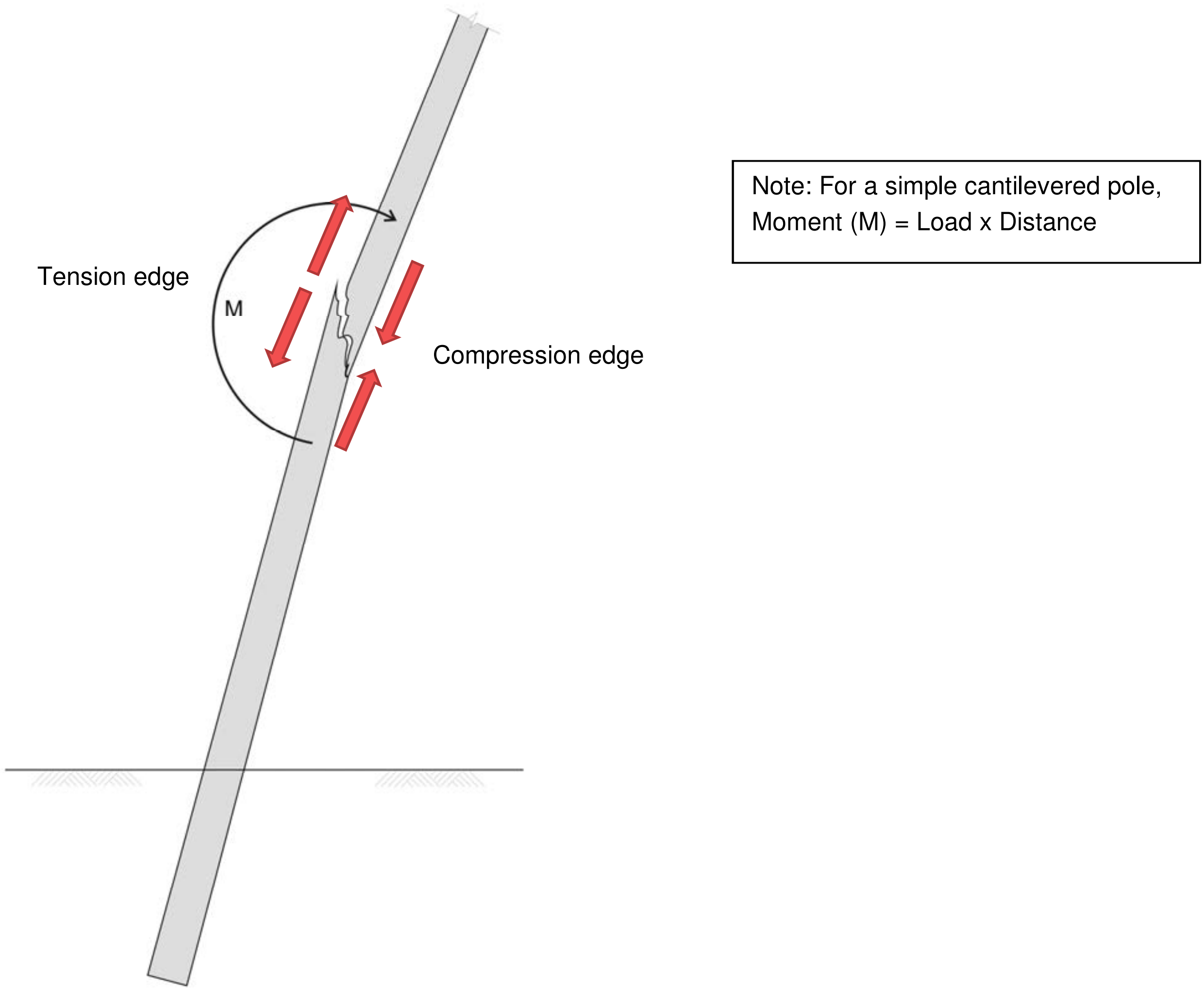


Figure 7 – Moment Direction on Pole viewed from the SW

7.0 Timber Properties, Bending Strength and Section Modulus

7.1 Timber properties

7.1.1 At the time of the inspection, due to constraints included in my Instructions as well as WH&S requirements of Energex, I was not able to obtain samples of sufficient size (approximately 2000 mm in length) from the Failed Pole that could have been evaluated by laboratory testing to obtain actual properties of the failed pole.

7.1.2 The disc on the pole identifies the pole species as being Spotted Gum (*Corymbia Maculata or C. Citriodora*). This also concurs with the Energex advice given in Ref. 12.1 which also gives the installed age of the pole as approximately 20 years.

7.1.3 However, a small sample (approximately 25 x 25 x 200 mm long) of the pole I removed from site was forwarded to Dr Jugo Ilic for microscopic examination and confirmation of the species. This identification confirmed the actual species as being Blackbutt (*Eucalyptus Pilularis*). Refer to the identification certificate in Appendix 2.

7.1.4 This being the case, I have discounted Spotted Gum being the species and have reviewed existing Blackbutt strength data from a range of sources including 'small clear' timber tests as well as from tests that have been conducted on a number of full size Blackbutt poles to enable an estimate of the relevant strength properties of the failed pole.

7.1.5 In estimating the relevant strength properties I have considered a number of factors that could influence this strength, including:

a. Any strength reducing characteristics (defects) present at the fracture location
b. The inherent strength of blackbutt
c. The effect of pole size (diameter) and
d. The effect of loss of strength with time for the installed pole.

7.2 Bending strength and modulus of rupture

7.2.1 A close inspection of the Failed Pole including near the fracture point that I undertook did not indicate any knots, shaving or other disturbance to the generally, natural round nature of the pole that would unduly influence strength. See Appendix 3, Photographs 18 and 20 that indicate the condition of the timber adjacent to the fracture location.

7.2.2 Blackbutt is classified as Strength Group S2 (unseasoned) and SD2 (seasoned) (Ref. 12.2). The corresponding stress grade for natural round, mature species of this Strength Group given in AS 1720.1 is F27 which has a characteristic bending value of 67 MPa. (Ref. 12.2).

7.2.3 The mean modulus of rupture for S2 species based on small clear testing given in AS 2878 (Ref 12.3) is 86 MPa.

7.2.4 From static 3 point bending tests on small clear samples of Blackbutt from 18 trees sourced from NSW and QLD, undertaken by CSIRO, (Ref. 12.4), it was found that the mean modulus of rupture was 86.9 MPa (12,600 lb/sq.in.).

7.2.5 Dr Geoffrey Boughton, TimberEd Services Pty Ltd, provided me with more recent bending strength test data on new full size Blackbutt poles which I reviewed and accepted as being reliable. The results of this bending strength data, normalized to poles of 250 mm diameter is as follows:

a. 77 tests
b. average Modulus of Rupture (MoR) 85.5 MPa (1 MPa = 1 N/mm^2 = 145 lb/in^2)
c. Coefficient of Variation (CoV) 19.9%
d. 5th percentile strength (5%ile) 62.8 MPa
e. Characteristic Value (CV) 60.1 MPa

7.2.6 The strength data in Paragraph 7.2.5 above is normalized to poles of 250 mm in diameter. The Failed Pole had a diameter significantly greater than this being approximately 465 mm at ground line and 362 mm near the tension edge of the fracture. Strength size reduction factors have been determined for timber poles, including Blackbutt (Ref 12.6). This information was reviewed and re-analysed by Dr

Geoffrey Boughton, TimberEd Services Pty Ltd, who found a 0.08 MPa reduction per mm diameter for poles > 250 mm diameter. I have considered and accepted this reduction.

7.2.7 Loss of strength with time (new poles vs those with in-service history) also requires consideration. The Ausgrid Design Manual (Ref 12.5.) (which in turn references AS/NZS 7000 – Overhead line design – Detailed procedures), suggests a time/strength loss factor being K_d = 0.85 being applicable to poles at 50 years service.

7.2.8 However, more recent strength loss data that was provided to me by Dr Geoffrey Boughton, TimberEd Services Pty Ltd, has been derived from pole testing as shown in Figure 8. From this, for Blackbutt, a strength loss factor approximately equal to 0.8 would appear more applicable for mean strength loss for poles that have been installed for 20 years. Alternatively, to convert the Characteristic Value (CV) (60.1 MPa) to a mean bending strength taking into account strength time loss would require the CV to be multiplied by approximately 1.1. I have considered and accepted this information.

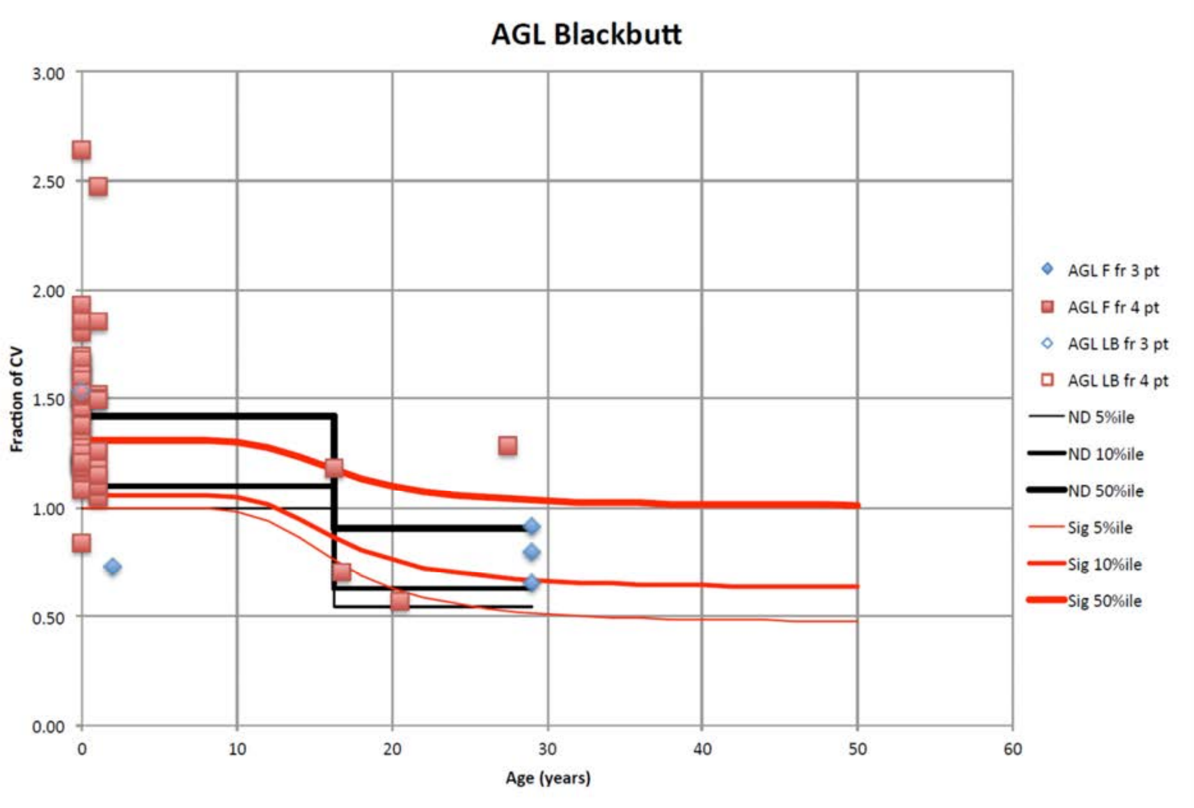


Figure 8 – Loss of Strength with Time – Blackbutt Poles (Source: Dr Geoffrey Boughton, TimberEd Services Pty Ltd)

7.2.9 From the above information, I consider the most reliable estimate of the strength of Blackbutt poles be based on the full size pole strength test data as given above, modified accordingly for pole size and loss of strength with time. From this it is estimated that at the time of the pole failure, the **average ultimate bending stress** the pole could sustain can be determined from:

$F_b = (F_m \times K_1 \times K_d) - K_s$

Where: F_b = average ultimate bending stress

F_m = Characteristic bending strength test value

K_1 = duration of load factor = 1.0 for loads of 5 seconds (and shock loads) to 5 minutes duration (Ref 12.2.)

K_d = time/strength loss factor = 1.1 (approx.) to adjust from Characteristic bending strength test value for a pole of 20 years age to an average bending strength value.

K_s = size strength reduction value = 17.2 MPa for 465 mm ground line diameter and 9.1 MPa for tension edge fracture diameter of 362 mm (based on 0.08 MPa reduction per mm diameter > 250 mm.)

Therefore at ground line for 20 year old Blackbutt poles of calculated diameter of 465 mm.:

$F_b = (F_m \times K_1 \times K_d) - K_s$

$= (60.1 \times 1.0 \times 1.1) - 17.2$

$\mathbf{= 48.91\ MPa}$

And at failure location for 20 year old Blackbutt poles of calculated diameter of 362 mm.:

$F_b = (F_m \times K_1 \times K_d) - K_s$

$= (60.1 \times 1.0 \times 1.1) - 9.1$

$\mathbf{= 57.01\ MPa}$

Note: A value of F_b = 55.8 MPa has been used as an estimated approximate value to simplify calculations (Appendix 4) over the length of the pole.

7.3 Effective X-Section of Pole for determination of Section Modulus

7.3.1 Under my direction, Mr Somerville undertook drill probing of the Failed Pole from the base to close to the top fracture location. This revealed unsound timber approximately 100 mm in from the outer surface. See Figure 9 for an indication of the location of the unsound timber.

7.3.2 Based on a closer review of the pole fracture location and visual assessment of the nature of the fracture, I concluded that a significant proportion of the center of the pole had 'brittle heart'. See Appendix 3, Photo 20 that identifies the location of the brittle heart.

7.3.3 Brittle heart typically develops in large mature hardwood trees and is characterized by having lower durability and significantly lower strength properties than adjacent mature outer heartwood. The brittle heart was most likely present in the pole from date of installation. Brittle heart is very 'short grained' and has minimal impact strength and typically fails with a 'carroty' type fracture. For this reason, I have discounted any potential bending strength contribution of this brittle timber to the overall pole strength at the fracture (or any point) point.

7.3.4 I have estimated from pole measurements and from Appendix 3, Photo 20 that the brittle heart has a diameter of approximately 190 mm at the fracture or slightly greater at the ground line.

7.3.5 For the purposes of calculation of estimated pole strength, I assumed a sound section (tapered cylinder) of pole with a wall thickness of 100 mm over the pole length.

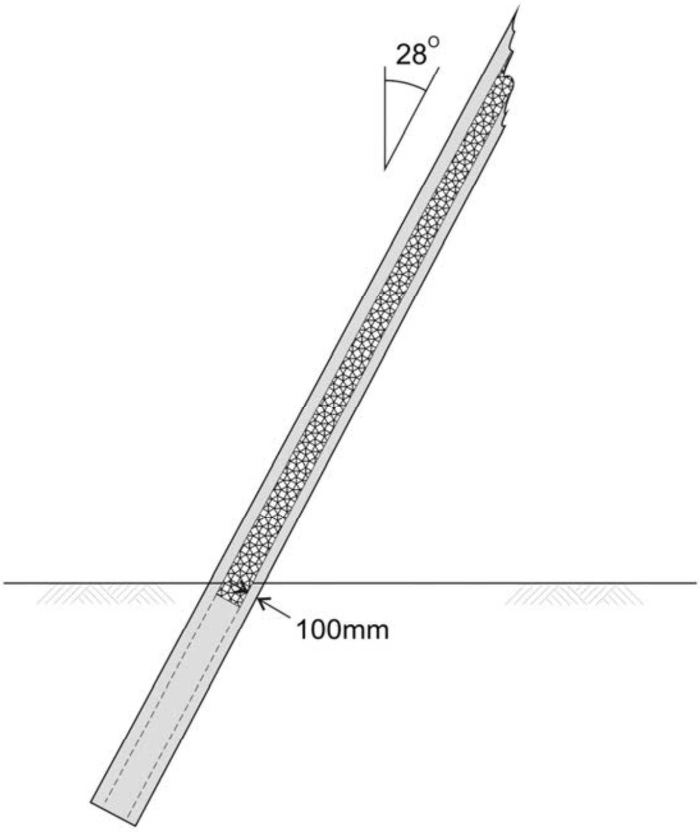


Figure 9 – Sound/Unsound timber (Unsound brittle heart timber indicated by dark hatching)

8.0 Load/Failure Scenarios

8.1 Location of fracture

The pole fracture is located between 4800 mm and 5800 mm measured up the leaning pole from the current mean ground line. This would be at approximately 127.85mAHD to 128.55mAHD to the bottom of the fracture if the pole were standing approximately vertically as originally installed. The pole identification disc is located 2450 mm from the current mean ground line when measured along the leaning pole. The distance from the disc to the fracture is therefore 2350 and 3350 respectively. Assuming the original pole length (17000 mm) and height to disc from pole butt (5500 mm) are correct (Appendix 5) then the assumed distance from the pole tip to the bottom of the fracture is approximately 9150 mm and to the top of the fracture, approximately 8150 mm.

8.2 Details related to possible failure scenarios

Details obtained from my investigation on the Failed Pole (Refer Paragraphs 4.1, 4.2, 4.5, 4.6 and 4.8) are:

a. The pole leaned towards approximately 124^0 N (i.e. to SE).

b. There was tension at the top of the fracture (NW side of pole) and compression at bottom (SE side of pole).

c. Fracture was caused by moments causing a concave curvature on SE side.

d. Fracture moment could be caused by a load to the SE above the fracture with cantilever action in the pole or towards the NW above or below the fracture provided the conductors and stays offered support to the top of the pole.

e. Bruising on the pole indicates a number of impacts on the NW side of the pole and none on the SE side of the pole.

f. Scouring around the pole may have contributed to some of its lean. Scouring is, I believe, to be generally highest in the highest velocity regions at side of pole and behind pole.

8.3 Possible load scenarios on pole that could have resulted in failure

8.3.1 I have considered a number of possible load scenarios that could have resulted in the pole failure. These are given below.

8.3.2 _Scenario 1_. Impact on the pole from a SE direction below the fracture location:

a. Absence of bruising on SE side of pole rules out impacts on SE side of pole that may have caused concave curvature to the SE if the conductors and stays are intact.

b. If flow was roughly towards the SE, flow forces could not have caused these actions

c. Implication of the above is that fracture was caused by loads above the failure point acting in SE direction.

8.3.3 _Scenario 2_. Water drag (including debris build up at or below peak flood level) on the pole itself caused by a peak flood level between 124.6m AHD and 129m AHD:

a. For the most severe case, for a peak flood level at 129.0 AHD at Pole 182127, the maximum flow height is at the approximate fracture point. This would also be the case for a flood level of 124.6 to 129mAHD.

b. Therefore, water drag would have put loads on the pole predominately below the fracture point.

c. I do not consider this could have caused the pole to fail in the manner it has.

8.3.4 _Scenario 3._ Loading by debris impacting the pole directly at or below the break:

a. A maximum water level at 129.0 AHD is at the fracture point and impact marks suggest a number of possible impacts well below fracture point.

b. I do not consider those impacts could have caused failure as curvature induced in the pole would be in the wrong direction to that which I determined.

8.3.5 _Scenarios 4a and 4b_. Loading due to debris impact with a stay on the Failed Pole (pole 182127): Under this scenario, there are two possibilities as described in paragraph a. and c. below -

a. _Scenario 4a_. Extra tension in the stay to the West would have pulled the pole in the wrong direction and caused fracture in a different orientation.

b. I do not consider this could have caused the pole to fail as it has.
c. *Scenario 4b*. Extra tension in the stay to the NE would have pulled the pole to NE, but pole leaned at almost 90 degrees to that direction
d. I do not consider this could have caused the pole to fail as it has.

8.3.6 *Scenarios 5a, 5b, and 5c*. Loss of stays to the Failed Pole (pole 182127):

a. These stays could have been removed by direct impact on the stays by debris or by undermining of the anchors to the stays or been broken by an overload. Once the stays were lost, then there are unbalanced forces on the top of the pole due to the normal tensions in the conductors to poles 182128 and 182126.
b. *Scenario 5a*. Loss of stay to the West. This would have caused an out of balance load of 15.18 kN (Refer Appendix 5) to the East, well within the capacity of the conductors but not high enough to cause failure of the pole
c. I do not consider this could have caused the pole to fail as it has.
d. *Scenario 5b*. Loss of stay to NE. This would have caused an out of balance load of 15.18 kN (Refer Appendix 5) to the SW, well within the capacity of the conductors but not high enough to cause failure of the pole. The tension in the conductors would have pulled the tip of the pole towards the SW and increased tension in the conductors to pole 182128 and reduced the tension in conductors to pole 182126 a little but not in the observed failure direction.
e. I do not consider this could have caused the pole to fail as it has.
f. *Scenario 5c*. Loss of stay to W and NE. This would have caused an out of balance load of 16.49 kN (Refer Appendix 5) to the SSE, well within the capacity of the conductors but not high enough to cause failure of the pole. The tension in the conductors would have pulled the tip of the pole towards the SE and decreased tension in the conductors to both poles 182126 and 182128 a little.
g. I do not consider this could have caused the pole to fail as it has.

8.3.7 *Scenario 6*. Impact with conductors (between pole182127 – 182126) and pole 182126:

a. Any impact with conductors between pole 182127 and 182126 or impacts with pole 182126 or its stays will produce extra tension in conductors fixed to pole 182127 in SW direction. Tension in these conductors could apply a load up to 3x22 kN (breaking capacity of conductors) near the top of pole 182127.
b. Such a load would compromise the NE stay and cause a tip load to the cantilever pole in SW direction. This load would have caused failure of the pole but in a SW direction.
c. This direction of failure was not observed so I do not consider this could have caused the pole to fail as it has.

8.3.8 *Scenario 7*. Impact with conductors (between pole 182127 – 182128) and pole 182128:

a. Any impact (such as debris snagging conductors) with conductors between pole 182127 and 182128 or impacts with pole 182128 or its stays will produce extra tension in conductors fixed to pole 182128 in E direction.
b. Tension in these conductors would apply a load up to 3x22 kN (breaking capacity of conductors near the top of pole 182127.

c. Such a load would compromise the W stay and cause a tip load to the cantilever pole in E direction. This load would have caused failure of the pole but in an E to SE direction as observed.

d. By considering the minimum clearance from ground to the conductors of 6.5 m to 8.5 m (Appendix 5) and the AHD to the conductors this would represent between poles 182127 and 182128 (approx. 129.35 to 130.1m AHD), if the peak flood AHD was in the vicinity of 129m AHD, then for the case of minimum conductor ground clearance (6.5 m) there would be less than 1.1 m clearance between the water and the conductors. For these assumptions and this scenario, floating debris could have quite conceivably snagged the conductors. The above estimates are based on my linear interpolation of the range of AHD's given in Appendix 5 for the ground at the poles being: the Failed Pole (Pole 182127), 122.5 – 123.2m AHD and Pole 182128, 123.2 – 124.0m AHD.

e. **I consider this to be a possible scenario that could have resulted in the pole failure**.

8.3.9 *Scenario 8*. Large debris striking the Failed Pole (pole 182127) directly above the break:

a. A possible scenario is large debris striking pole 182127 directly, with a protrusion that extended beyond to be able to impact pole above the break or else debris pushing pole over and riding up the pole to place a large load on the tip.

b. This scenario could place enough load on the pole to break the stays to pole 182127 and cause failure of the pole.

c. However the pole would have had to have been pushed down so that the debris was near the tip of the pole and this would have been at a much flatter angle than the pole is currently at.

d. It would then have had to rebound to the position in which it is seen today. That amount of rebound is unlikely.

e. **I consider this to be a possible, but less likely scenario for the pole failure**.

8.3.10 *Scenario 9:* Embankment on south eastern side of the pole that may have restrained the pole.

a. I also considered or undertook a sensitivity check to address additional pole restraint options as detailed in Instruction Letter #2 (Appendix 5) being:

i. Assuming the actual ground level around the pole or ground level of an adjacent embankment to the SE of the pole at the time of failure was 2.0 m to 5.0 m higher than used in the above calculations (i.e. a pole embedment depth or adjacent ground level of 4.5 to 7.5 m. i.e. ground at approximately 125mAHD to 128mAHD).

This check did not materially alter the failure location or stresses induced in the pole at the failure location.

ii. Assuming that at the time of failure, the pole was restrained by the top of an embankment to the SE of the pole, adjacent to the pole and being significantly higher (for example 130m AHD) than the level at point of fracture of the pole.

b. This scenario would alter the probable location of maximum stresses and failure point in the pole resulting in failure significantly above the current actual fracture location.

c. Also, the current lean on the pole is approximately 62^0 (90 – 28 deg) to the horizontal. For the assumptions given in Instruction Letter #2 (Appendix 5) regarding the slope of the embankment being 35^0 to the horizontal, the pole would have had to have rotated (leaned) an additional 27^0 or greater for the embankment to have provided sufficient restraint on the pole for this to have contributed to failure. Refer Figure 10.

d. I do not consider this option to have caused or contributed to the pole failing at the point observed.

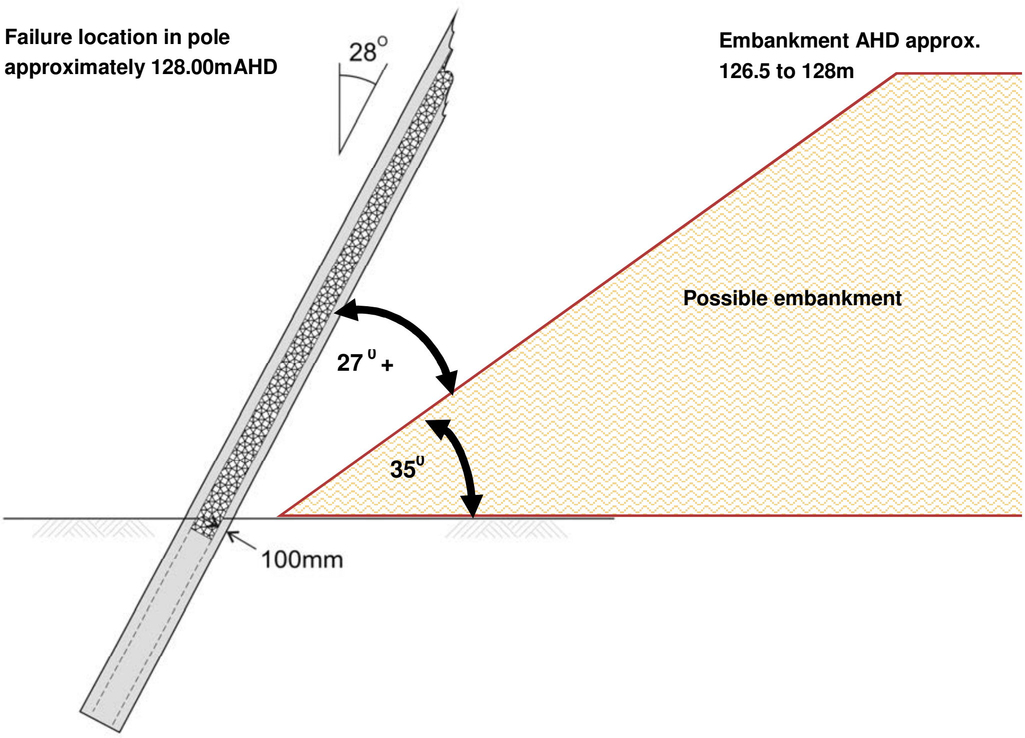


Figure 10. – Possible embankment

8.3.11 Of the above possible scenarios and the fact that the pole fracture location is well above the current ground level (and possible ground level at the time of the flood) I consider that a sudden loading event, such as described in Paragraph 8.3.8, *Scenario 7*. above (possibly in combination with a build-up of flood debris on attachments to the pole etc), applied in an approximately SE direction, resulted in the failure. This likely scenario is also supported by the pole analysis described in Paragraph 8.4 and Appendix 4.

8.4 Pole Analysis

8.4.1 A bending capacity structural analysis of the pole was undertaken to try and establish if the maximum stresses induced in the pole aligned with the nature and location of the pole fracture for the likely possible failure scenarios.

8.4.2 A static and dynamic moment analysis (Refer Appendix 4 for spreadsheet of calculation) based on the likely failure scenario was undertaken by Dr Geoffrey Boughton, TimberEd Services Pty Ltd, which I have reviewed and accept. The following assumptions and criteria were used for this analysis:

a. Assumed load applied at the X-arms

b. 'Banana' conductors have a breaking strength of 22.7 kN. (Ref 12.7) However, because of the connections at the insulators etc, it could be less than that value. A 'Banana' load divided by a factor of 2, indicates there is enough load that could be imparted to pole 182127 via the conductors to exceed the 20 year average strength of blackbutt poles at the failure location.

8.4.3 The strength analysis of the pole is as follows:

a. Measured taper of 17.4 mm/m length of pole

b. Assumed that the depth of sound wood is uniform up the pole

c. Assumed 100 mm of sound wood on the outside of the pole

d. Assumed a pulse load of duration 1/10th second for the dynamic analysis

e. 68.1 kN (breaking load of 3 'bananas')

f. Assumed that the pole length was as marked on drawings =17 m

g. Assumed depth of embedment 1/10th length of pole plus 800 mm = 2.5 m

h. Assumed cross arm location 0.6 m below pole tip

i. Size reduction factor - for hardwoods -0.08 MPa/mm.

8.4.4 From this analysis it is estimated (calculated) that the peak calculated dynamic and static moment was at approximately 7.0 m (approximately 129mAHD) above ground as shown in Appendix 4, Sheet 2. Also,

a. Peak moments were pretty consistent over the region 5 m to 7.5 m above ground line.

b. The fracture region 5 m to 6m (approximately 128mAHD) is in the area with maximum stresses in the pole under shock or static loading.

c. Thus it is not unexpected that the pole failed at this location despite there not being an identifiable defect there.

9.0 Discussion

9.1 Location of failure point on the Failed Pole (Pole 182127)

9.1.1 Timber utility (power) poles can fail at many locations along their length depending upon numerous factors including:

a. Degradation that may have occurred to the timber at different locations such as decay or termite damage to the timber. Poles that have been snapped off or sheared off at their base (ground line or a bit above or below ground line) are often seen and can usually be related to decay/termite damage that reduces the effective strength (sound cross sectional area) of the pole at or near the ground. Conversely, pole breaks are often seen well above ground line following investigations of line failures resulting from high wind events such as cyclones.

b. The point of load application that causes the pole to fail, such as close to the ground, where for example, a vehicle may have collided, to up near the tip of the pole where the overhead loads from conductors attached or where wind loads are transmitted to the pole. In a flood scenario, there are other load point possibilities including debris loads on the pole itself or from debris loads applied through attachments to the pole towards its top.

c. The inherent strength resistance of the pole at any location along its length.

d. Natural round poles, the same as the trees they are obtained from, are tapered along their length having larger diameters at their base/butt than at their top. The greater the diameter of sound timber, the greater the strength of the pole.

e. Location of maximum stress in the pole (as a result of point/s of application of load/s that induced failure) and inherent strength resistance of pole at the failure location.

9.1.2 Figure 11 provides a simple illustration of the inherent strength of the pole (assuming constant timber properties along its length) to the bending moment induced in the pole (assuming the pole is a simple cantilever with a load applied close to top of pole).

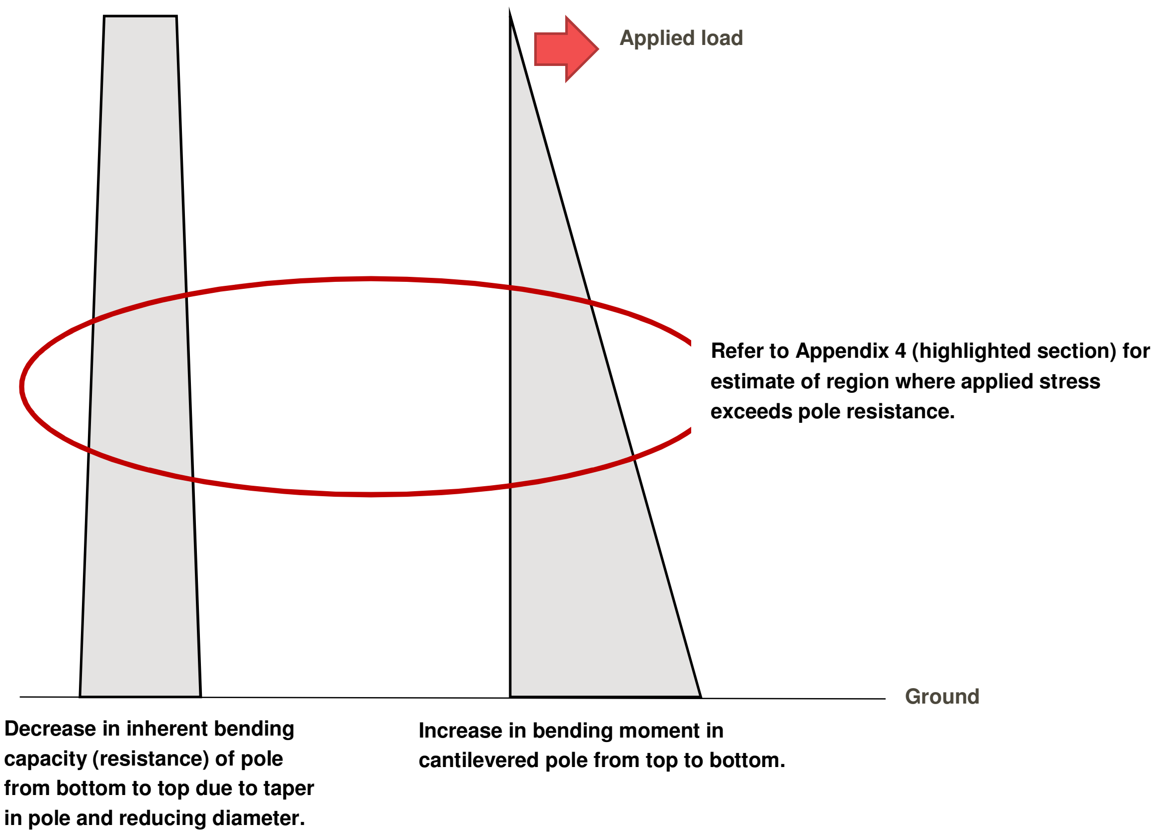


Figure 11 – Illustration of Bending Capacity of Pole vs Applied Bending Moment

10.0 Summary of Findings

10.1 The Failed Pole (Pole 182127) failed at greater than 4800mm (vertical projection from current position) above the ground level from measurements recorded at time of inspection.

10.2 The direction of lean on the Failed Pole and also the orientation of the fracture roughly align with the resolved attached conductor directions.

10.3 The moment induced in the pole at the failure was in a clockwise direction when viewed from the SW and also aligns with the lean of the pole direction.

10.4 The dynamic or static stress analysis indicates that the maximum stresses in the pole align closely with the fracture location.

10.5 For a simple cantilevered pole (un-stayed), a point load of approximately 31kN (3.1 tonne approx.) or greater, applied at or near the pole top (X-arms) would be required to cause pole failure. Similarly if a point load of approximately 62kN (6.2 tonne approx.) or greater, was applied at about the mid-point between the pole top (X-arms) and the fracture location, it would cause pole failure

10.6 An increase in the height of the ground level at the base of the pole from time of installation to at time of flood event of 200 to 300 mm would not have had any significant impact on the above estimates as the maximum stresses induced in the pole are well above the ground line.

10.7 From the evidence and measurements obtained from my site inspection together with my assessments and analysis, I am not able to conclude if there has been any change in the level of ground surrounding the pole from time of initial pole installation to time of flood event, or thereafter.

10.8 I consider the most likely scenario for failure of the pole to be flood debris impact with the conductors between the Failed Pole (Pole 182127) and Pole 182128 and/or directly on Pole 182128 and, a less likely scenario being, large debris striking Pole 182127 directly above the break.

11.0 Qualifications to Report

11.1 There are many unknowns and uncertainties relating to the assessments undertaken and also possible scenarios that could have resulted in the failure of Pole 182127 or associated infrastructure connected to this pole.

11.2 These factors include, but are not limited to:

a. Obtaining more reliable or confirming pole timber strength properties. These could be obtained if sections of the pole could be obtained for laboratory strength testing.
b. Eye witness or video reports that described or show the actual failure of the pole during the flood event.
c. Additional geotechnical information relating to the current location of the butt of the pole with respect to its original location (embedment depth etc.) or other embedment depths or possible pole restraint locations.
d. More definitive advice on the actual minimum distance between conductors and ground level at time of the flood
e. More definitive advice on direction of flood flow and maximum flood levels between Poles 182126 and Pole 182128

11.3 This report provides my best estimates of likely failure scenarios based upon measurements and observations on site, together with information provided by other parties including Energex (Ref 1) and the Commission (Appendix 5).

11.4 Should additional facts or information become available, I reserve the right to re-consider my findings.

12.0 References

12.1 Statement of Ronald Barbagallo, Energex, 25 June 2015

12.2 AS 1720.1 – 2010. Timber Structures, Part 1 Design Methods, Standards Australia 2010.

12.3 AS/NZS 2878 :2000. Timber – Classification into Strength Groups. Standards Australia 2000.

12.4 The Mechanical Properties of 174 Australian Timbers. E Bolza and N H Kloot. Technical Paper 25. CSIRO , Australia 1963

12.5 Ausgrid Design Manual NS 220 inc. amendments 2013 and 2014

12.6 Amanda Yeates et al. Determination of Characteristic Stresses and Residual Life Expectancies of Preservative Treated Hardwood and Softwood Poles. Queensland Government, Department of Primary Industries, Forest Research Institute. March 2003.

12.7 Dulhunty Power Catalog, Section M.

13.0 Acknowledgments

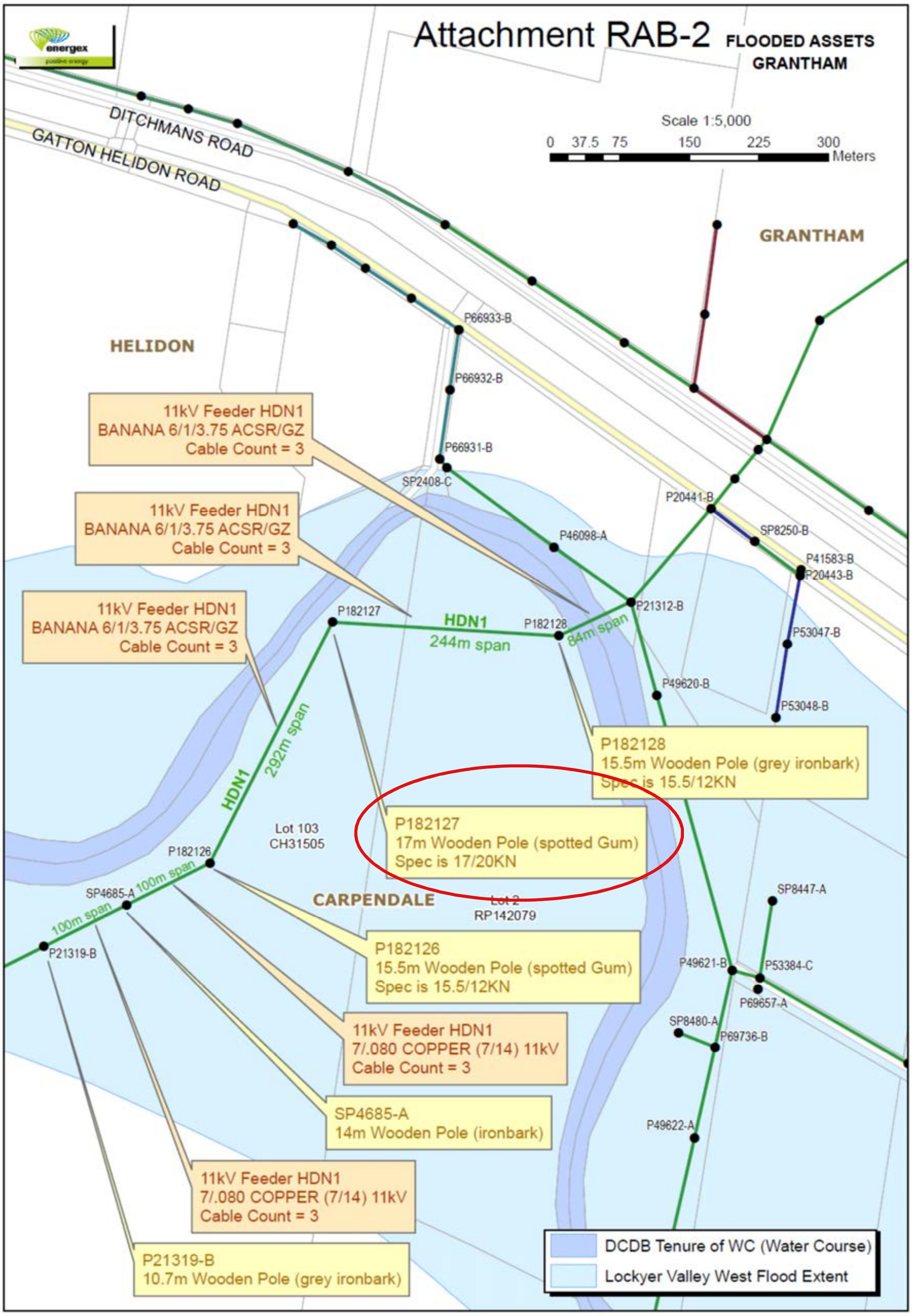
The experience, knowledge, advice and structural analysis contribution provided by Dr Geoffrey Boughton, TimberED Services, Pty Ltd, that enabled completion of this report is gratefully acknowledged.

[redacted]

Colin MacKenzie
MacKenzie Consulting

27 July 2015

Appendix 1 – Energex Site Location Showing Location of Investigated Failed Pole


Attachment RAB-2
FLOODED ASSETS
GRANTHAM
energex
Scale 1:5,000
0 37.5 75 150 225 300 Meters
DITCHMANS ROAD
GATTON HELIDON ROAD
GRANTHAM
HELIDON
CARPENDALE
P66933-B
P66932-B
P66931-B
SP2408-C
P46098-A
P20441-B
SP8250-B
P41583-B
P20443-B
P21312-B
P53047-B
P49620-B
P53048-B
P182127
P182128
HDN1
244m span
84m span
292m span
HDN1
P182126
100m span
SP4685-A
100m span
P21319-B
Lot 103
CH31505
Lot 2
RP142079
SP8447-A
P49621-B
P53384-C
P69657-A
SP8480-A
P69736-B
P49622-A
11kV Feeder HDN1
BANANA 6/1/3.75 ACSR/GZ
Cable Count = 3
11kV Feeder HDN1
BANANA 6/1/3.75 ACSR/GZ
Cable Count = 3
11kV Feeder HDN1
BANANA 6/1/3.75 ACSR/GZ
Cable Count = 3
P182128
15.5m Wooden Pole (grey ironbark)
Spec is 15.5/12KN
P182127
17m Wooden Pole (spotted Gum)
Spec is 17/20KN
P182126
15.5m Wooden Pole (spotted Gum)
Spec is 15.5/12KN
11kV Feeder HDN1
7/.080 COPPER (7/14) 11kV
Cable Count = 3
SP4685-A
14m Wooden Pole (ironbark)
11kV Feeder HDN1
7/.080 COPPER (7/14) 11kV
Cable Count = 3
P21319-B
10.7m Wooden Pole (grey ironbark)
DCDB Tenure of WC (Water Course)
Lockyer Valley West Flood Extent

Appendix 2 – Pole Species Identification

KNOW YOUR WOOD
19 Benambra Street, South Oakleigh,
Victoria 3167, AUSTRALIA
Phone: 03 95127523
Mobile: 0499 300 208
Email:knowyourwood1@gmail.com
Provider of wood identification services

29th June, 2015

WOOD IDENTIFICATION RESULTS

Mr Colin MacKenzie
MacKenzie Consulting
PO Box 584,
The Gap, QLD, Australia 4061

Dear Colin,

Re: Assessment one wood specimen from a pole subject of a Commission of Inquiry, Grantham, QLD: Your request – 27 Jun, 2015

Following microscopic examination, in my opinion the structure of the wood specimens is consistent with[1]:

Sample	Scientific name	Commercial or Trade name + Remarks
Pole 182127	*Eucalyptus pilularis*	BLACKBUTT (AD density <1000, approximately 900kgm^{-3}; 7mm sapwood zone was evident with several radial weathering checks)

I hope the information will help with your evaluation process.

Best regards,

Jugo Ilic

Jugo Ilic MSc, Dr(Forest)Sc, FIAWSc

[1] *Disclaimer*: The content of this letter is provided in good faith and whilst Dr Jugo Ilic has endeavoured to ensure that the information contained in it is correct and accurate at the time of preparation, he does not accept any liability arising from its use whether provided directly by the above named client or indirectly from the client providing it to a third party in this or any other format.

Appendix 3 – Photographs to Report

Photo 1 – View of pole from the North. Note paint dot indicating North.

Photo 2 – View of pole from the West

Photo 3 – View of pole from the East

Photo 4 – Pole Identification # and Current leakage test point (highlighted)

Photo 5 – Pole Identification Disc.

Photo 6 – Bruising/damage to pole on NW face highlighted

Photo 7 – Damage (possible shelling) to pole on West face highlighted

Photo 8 – Debris around base of pole

Photo 9 – Evidence of possible top of old fence post adjacent to post

Photo 10 – Digitally enhanced picture of base of pole indicating height of slight colour variation against pole approximately 1100 mm up from ground line.

Photo 11 – Scouring at front of post to a depth of approx. 250 mm from actual ground. Note also wire and log debris etc.

Photo 12 – Scouring to rear (SE) of post to approximately 800 mm from actual ground

Photo 13 – Damage to lower portion of pole

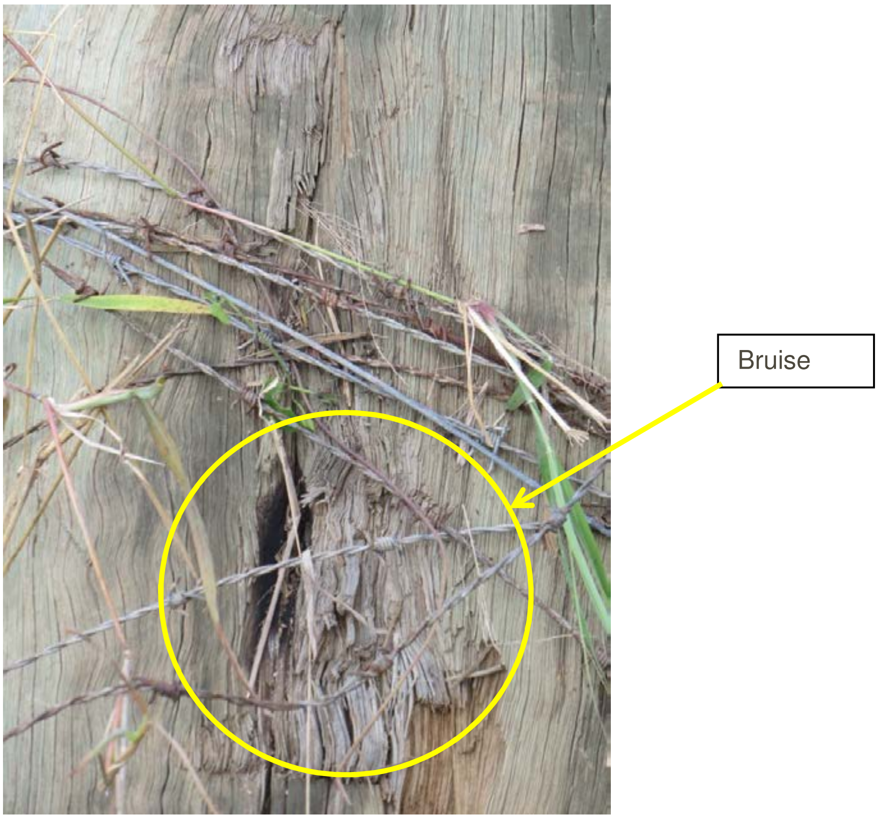


Photo 14 – Close up of damage to lower portion showing bruising

Photo 15 – Damage to middle section of pole

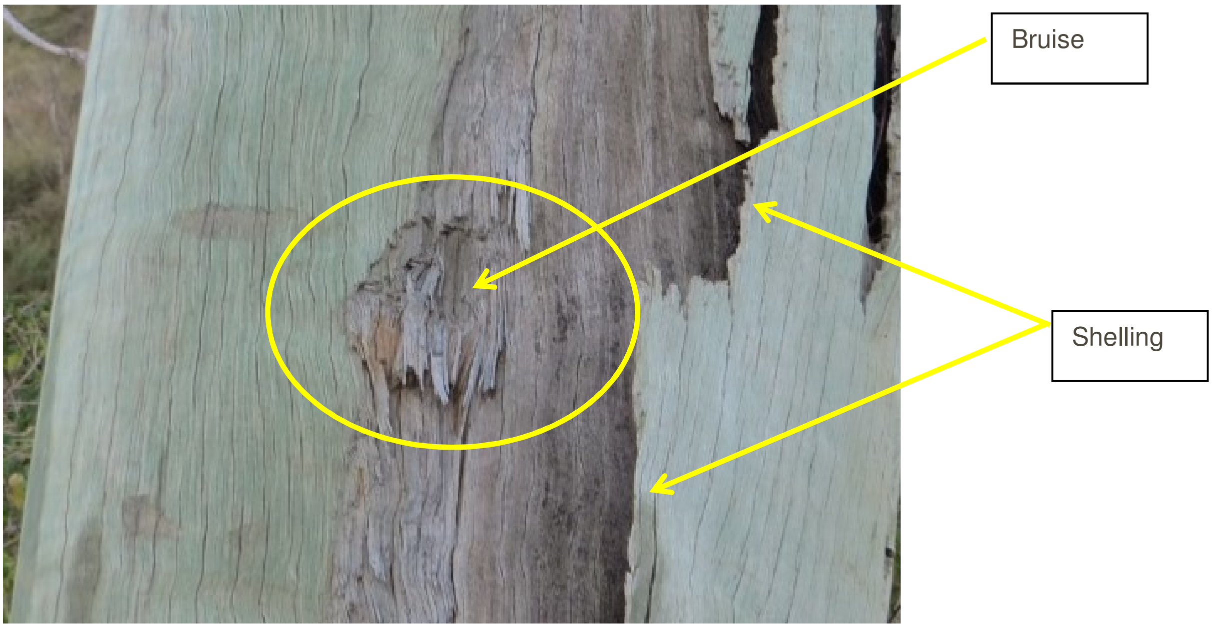


Photo 16 – Close up of bruising damaged middle section of pole and typical shelling

Photo 17 – Damage to upper section of remaining pole. Appears to be shelling

Photo 18 – Pole fracture viewed from the SW

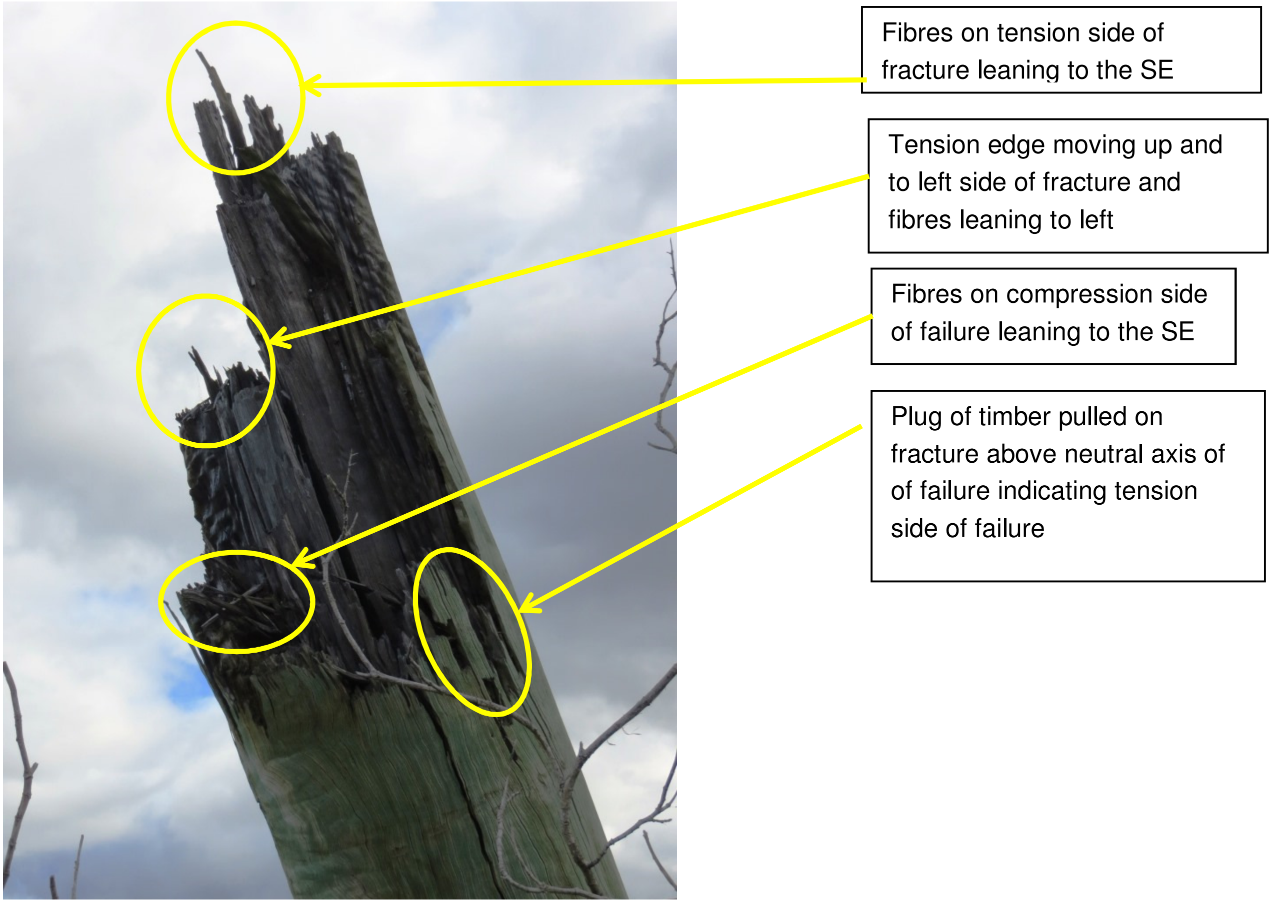


Photo 19 – Pole fracture from the NE

Photo 20 – Close up of pole fracture from the east showing signs of compression of fibres.

Semi circular shape of brittle heart on underside of fracture. A similar amount (size) of brittle heart would be located on top side of fracture.

Photo 21 – Indication of brittle heart

Appendix 4 – Pole Analysis – Spreadsheet

Page 1

BLACKBUTT					
Conductor	banana 6/1/3.75 ACSR/GZ				
No Conductors	3				
Breaking load conductors	22.7	kN	Dulhunty Power catalog		
			Section M (Energex 22.8)		
Pole length	17	m			
species	blackbutt				
embedment	2.5	m			
GL dia	464	mm			
likely taper	17.4	mm/m			
Fracture above GL	5	m			
fracture dia	377				
depth good wood (GW)	100	mm			
Zeff	4716060.72	mm3			
Cross arm to top	0.6	m	assumed		
lever arm					
cross arm to fracture	8.9	m			
cross arm to GL	13.9				
age in 2011	20				
speed of sound in HW	3960	ms-1			
assumed duration of shock loading	0.1	s			
time to fracture	0.00224747	s			
attenuation	0.97801926		(Not that significant)		
20 year old AGL spotted gum					
New pole CV	60	Mpa	From GNB 2015 EA study		(Ref 5)
Age factor avg strength	1.1		From GNB 2015 EA study		(Ref 5)
Avg AGL strength	66	Mpa	250 mm poles		
with size reduction	55.84	Mpa	From GNB 2015 EA study		
			at fracture location (Ref 5)		

Page 2

	BLACKBUTT										
		Duration of shock loading					T	0.1			
		Resolved total applied force on pole 182127 in direction of failure from conductors from adjacent poles 182128 and 182126					P	63.7617454		(3x22.7x0.936)	
						static load stress				impact load stress	
Dist above GL (m)	Lever arm (m)	Taper (mm/m)	Diameter (mm)	Depth good wood	Section Modulus (Z) (mm3)	Applied stress M/Z	Factor to Avg AGL strength	time from top	attenuation	dyn M/Z	Factor to Avg AGL strength
0	13.9	17.4	464	100	8779624.83	100.95	1.81	0.003510	0.966089	97.53	1.75
0.5	13.4	17.4	455.3	100	8349990.57	102.32	1.83	0.003384	0.967269	98.98	1.77
1	12.9	17.4	446.6	100	7931993.64	103.70	1.86	0.003258	0.968452	100.43	1.80
1.5	12.4	17.4	437.9	100	7525619.01	105.06	1.88	0.003131	0.969638	101.87	1.82
2	11.9	17.4	429.2	100	7130850.45	106.41	1.91	0.003005	0.970826	103.30	1.85
2.5	11.4	17.4	420.5	100	6747670.36	107.72	1.93	0.002879	0.972018	104.71	1.88
3	10.9	17.4	411.8	100	6376059.7	109.00	1.95	0.002753	0.973212	106.08	1.90
3.5	10.4	17.4	403.1	100	6015997.72	110.23	1.97	0.002626	0.974409	107.41	1.92
4	9.9	17.4	394.4	100	5667461.9	111.38	1.99	0.002500	0.975610	108.66	1.95
4.5	9.4	17.4	385.7	100	5330427.65	112.44	2.01	0.002374	0.976813	109.83	1.97
5	8.9	17.4	377	100	5004868.13	113.39	2.03	0.002247	0.978019	110.89	1.99
5.5	8.4	17.4	368.3	100	4690753.96	114.18	2.04	0.002121	0.979228	111.81	2.00
6	7.9	17.4	359.6	100	4388052.9	114.79	2.06	0.001995	0.980441	112.55	2.02
6.5	7.4	17.4	350.9	100	4096729.53	115.17	2.06	0.001869	0.981656	113.06	2.02
7	6.9	17.4	342.2	100	3816744.84	115.27	2.06	0.001742	0.982874	113.30	2.03
7.5	6.4	17.4	333.5	100	3548055.72	115.01	2.06	0.001616	0.984095	113.18	2.03
8	5.9	17.4	324.8	100	3290614.46	114.32	2.05	0.001490	0.985320	112.65	2.02
8.5	5.4	17.4	316.1	100	3044368.08	113.10	2.03	0.001364	0.986547	111.58	2.00
9	4.9	17.4	307.4	100	2809257.64	111.22	1.99	0.001237	0.987778	109.86	1.97
9.5	4.4	17.4	298.7	100	2585217.28	108.52	1.94	0.001111	0.989011	107.33	1.92
10	3.9	17.4	290	100	2372173.28	104.83	1.88	0.000985	0.990248	103.81	1.86
10.5	3.4	17.4	281.3	100	2170042.78	99.90	1.79	0.000859	0.991487	99.05	1.77
11	2.9	17.4	272.6	100	1978732.36	93.45	1.67	0.000732	0.992730	92.77	1.66
11.5	2.4	17.4	263.9	100	1798136.24	85.10	1.52	0.000606	0.993976	84.59	1.51
12	1.9	17.4	255.2	100	1628134.27	74.41	1.33	0.000480	0.995225	74.05	1.33
12.5	1.4	17.4	246.5	100	1468589.3	60.78	1.09	0.000354	0.996477	60.57	1.08
13	0.9	17.4	237.8	100	1319344.12	43.50	0.78	0.000227	0.997732	43.40	0.78
13.5	0.4	17.4	229.1	100	1180217.64	21.61	0.39	0.000101	0.998991	21.59	0.39
14											
14.5											
15											
15.5											
	Note: Factor to average Above Ground Line (AGL) strength = applied stress/estimated strength										

Appendix 5 – Letters of Instructions

Grantham Floods Commission of Inquiry

Reference number: TF/15/13780

Mr Colin MacKenzie
Mackenzie Consulting
By email: colin@timberexpert.com.au

Dear Mr MacKenzie

Engagement as a timber expert for the Grantham Floods Commission

The Grantham Floods Commission of Inquiry (the **Commission**) has been established to make a full and careful inquiry with respect to the flooding of the Lockyer Creek between Helidon and Grantham on 10 January 2011.

The scope of the Commission's terms of reference includes an inquiry in relation to the Grantham quarry and the impact the existence or breach of that quarry had on the flooding at Grantham.

Scope of your engagement

The Commission has engaged you to provide an expert opinion regarding the damage to a timber utility power pole (pole 182127) (the **Broken Pole**). The Broken Pole formed part of a line of power poles on Lot 103 on CH 31505. We have previously provided you with an aerial photograph in which the location of that power pole and its proximity to the quarry is indicated and a copy of this photograph is attached as Figure 1.

Your opinion is sought as to the likely cause or causes of failure of the Broken Power Pole. In providing your opinion, you are requested to:

1. identify the type of timber, probable design and construction and likely condition of the Broken Pole prior to the January 2011 flood;

2. identify the probable properties of the Broken Pole prior to the flood including, the ultimate (failure) bending moment strength or equivalent yield stress;

3. determine the estimated horizontal force that would be required to break the Broken Pole (as observed);

PO Box 15185
CITY EAST QLD 4002

Telephone
(07) 300 39451

Email
mailbox@granthaminquiry.qld.gov.au

4. determine, to the extent it is possible to do so, the extent to which any (or all) of the following contributed to the failure of the Broken Pole:

   (a) the likely direction of water flow;
   (b) the likely velocity of water flow;
   (c) the level of the Broken Pole above surface level;
   (d) the heights of water flow; and
   (e) the floating debris that may have been carried on the water flow.

In forming your expert opinion, you are requested to undertake all those investigations you consider necessary including, but not limited to:

1. undertaking site inspections of the Broken Pole, together with sampling and testing of the timber as required;
2. reviewing applicable standards, specifications and other literature as relevant to the Broken Pole; and
3. examining historical records and further information provided by Energex.

To clarify item 1, the Commission requests that the Broken Pole be inspected in-situ and that any sampling of the timber occur on the above ground part of the stub only. The Commission instructs you not to extract or remove the Broken Pole from its location. The Commission recognises that this instruction may limit your ability to determine the current embedment depth of the pole and the structural capacity of the power pole as a whole.

Assumptions to be adopted

The Commission notes that you have previously been provided with a statement from Mr Ronald Andrew Barbagallo, an engineer from Energex, regarding the Broken Pole and the line of power poles on which the Broken Pole has been placed (**Mr Barbagallo's Statement**).

The Commission also notes that Energex has previously advised that:

1. the top section of the Broken Pole and any associated cross-arm attachments and/or conductors are no longer available for inspection;

2. the test point for its power poles are normally 2.7m above ground level;

3. the ID disc for its power poles have been installed 5.5m from the butt of the pole. The height of the disc from ground level is 5.5m less the sinking depth of the pole;

4. the sinking depth of the Broken Pole is approximately 2.5m;

5. the pole number for any timber poles must be a minimum of 2.7m from the ground to comply with Energex standards;

6. Energex standards are given in the Design Manual which complies with Australian Guidelines for Design and Maintenance of Overhead lines (as amended from time to time);

7. it is reasonable to assume that:

   (a) the pole top to king bolt of top cross-arm is 150mm in length;
   (b) the king bolt of top cross-arm to king bolt of bottom cross-arm is 850mm in length;
   (c) the king bolt of bottom cross-arm to the stays is approximately 300mm in length; and

8. ignoring any external actions (such as wind or floods) on pole 18127 and assuming conductors from this pole to adjacent poles in line (P18126 and P18129) are in place, the horizontal tip load (action) to be applied to pole 18127 and the resolved direction of the load (angle to the North) are:
   (a) (if the stay to the North East was broken only), 15.18kn at 207.2 degrees from the North;
   (b) (if the stay to the West was broken only), 15.18kn at 93 degrees from the North; and
   (c) (if both stays were broken at the same time), 16.49kn at 150.1 degrees from the North.

   (Such assumptions have been given on the basis of 3 x 6/1/3.75 ACSR conductors strung at Table 65 terminating on each cross arm at 15°C and in a no wind condition).

To the extent you consider it necessary, you are instructed to adopt the information provided by Energex in Mr Barbagallo's Statement and listed above as assumptions in forming your opinion.

In forming your opinion, you are instructed to undertake sensitivity testing based on a range of different:

1. flood height levels at the location of the Broken Pole, but assuming that the range was no less than base of the pole and between 124.6mAHD and 129mAHD;

2. flood flow directions at the location of the Broken Pole, but assuming the primary direction of flow from Poles 182128 to 182126 is between 90 degrees true North to 160 degrees true North; and

3. surface levels around the base of Pole 182128 to 182126, but assuming the surface level around the base of:

Page 3 of 6

(a) Pole 182128, was between 123.2m AHD and 124mAHD;
(b) Broken Pole, was between 122.5mAHD and 123.5mAHD; and
(c) Pole 182126, was between 127.5mAHD and 127.66mAHD;

4. velocities of flood flow at the location of the Broken Pole, but assuming that the velocity was no less than 0.1m/s and no more than 10m/s.

Form of your expert opinion

The Commission requests that your opinion be provided in the form of an expert report addressed to the Commission and signed by you. That report must include the following information:

1. your qualifications;
2. all material facts and assumptions, whether written or oral, on which your report is based;
3. references to any literature or other material relied on by you to prepare your report;
4. for any site inspection, examination or experiment conducted, initiated or relied upon by you to prepare your report:
   (a) a description of what was done;
   (b) whether the inspection, examination or experiment was done by you or under your supervision;
   (c) the name and qualifications of any other person involved; and
   (d) the result;
5. if there is a range of opinion on the matters dealt with in the report, a summary of the range of opinion, and the reason why the expert adopted a particular opinion;
6. a summary of the conclusions reached by you; and
7. a statement about whether access to any readily ascertainable additional facts would assist you in reaching a more reliable conclusion.

The Commission also requests that your report includes suitable drawings, maps and photographs to explain your findings.

Finally, we advise that the Commission has scheduled public hearings to take place at the Gatton Cultural Centre commencing 20 July 2015 and continuing to 21 August 2015. We ask that you ensure that you are available during this period as we may require you to give evidence regarding your report during these hearings. In the event that this is required, we will provide you with as much notice as possible.

We look forward to receiving your report in due course.

Kind Regards

Joanne Paterson
Director
Grantham Floods Commission of Inquiry

16 July 2015

Figure 1. Google Earth image of Lot 103 on CH31505 showing the approximate locations of remaining power poles as circles. Arrow points to the broken pole.

Grantham Floods Commission of Inquiry

Reference number: *DOC/15/127412*

Mr Colin Mackenzie
Mackenzie Consulting
By email: colin@timberexpert.com.au

Dear Mr Mackenzie

Instruction letter #2 - Engagement as a timber expert for the Grantham Floods Commission

We refer to our previous letter of instructions dated 16 July 2015.

In that letter, we confirmed your engagement to provide an expert opinion regarding the likely cause or causes of failure to a timber utility power pole (pole 182127).

In addition to the matters set out in that letter, we request that you consider the hypothesis that the failure of the pole was caused or contributed to by an embankment located to the south-east side of the pole, and to the west of the quarry pit. That is, we ask that you express your opinion as to:

(A) whether the failure of the pole at the point observed could have been caused or contributed to by a nearby embankment, assuming the embankment was as described below;

(B) if you do not consider that the nearby embankment, as described below, could have caused or contributed to the failure of the pole, whether if the embankment was higher than indicated in 2 (c) below, it might have caused or contributed to the pole failing at the point observed.

To assist you in forming an opinion:

1. we **attach** an aerial photograph of the pole 182127 taken in July 2010 showing the location of the power pole in connection with the embankment to the south east of the pole (**2010 Photograph**).

2. we request that you make the following assumptions, for the purposes of (A), above, insofar as it concerns pole 182127 prior to the flood:

(a) the location of the pole 182127 and its proximity to the embankment to the south east of the pole is the same as is shown in the 2010 Photograph;

(b) the surface level of the base of pole 182127 was between 122.5m AHD and 123.5m AHD;

(c) the peak of the embankment to the south east of the pole was between 126.5m AHD to 128m AHD;

(d) the distance between pole 182127 and the base of the embankment to the south east of the pole was between 1m to 2m;

(e) at its peak, the embankment to the south east of the pole was between 7m to 8m from the power pole; and

(f) the embankment to the south east of the pole was a slope which, on the side of the power pole, was at an angle of 35 degrees.

We request that you include your opinion on the above matters in the expert report that is to be delivered in answer to our first set of instructions.

We look forward to receiving your report in due course.

Yours sincerely

Joanne Paterson
Director
Grantham Floods Commission of Inquiry

27/ 7 /2015

Attachment: Aerial photo from 2010 supplied by David Starr

http://maps.au.nearmap.com/ 23/07/2015

Appendix 6 – Curriculum Vitae

Curriculum Vitae: Colin MacKenzie

Colin Elliott MacKenzie:	Dip. Eng. (Civil) F.I.E. Aust. C.P.Eng. RPEQ, NPER
Current Positions:	Technical Consultant to: Timber Queensland Ltd (formerly Timber Research and Development Advisory Council) and, Principal MacKenzie Consulting

Colin was educated in Melbourne and graduated with a Diploma of Engineering (Civil) from Caulfield Institute of Technology in 1974. He was employed as a Technical Assistant and Technical Officer with the CSIRO, Division of Forest Products (later Division of Building Research) for a 7 year period up until 1976, where he gained extensive experience in timber technology, timber and materials testing and laboratory practices and procedures.

In 1976 he joined the Timber Research and Development Advisory Council of Queensland (TRADAC) as their Engineer and later, Technical Director. In 2003, TRADAC and the Queensland Timber Board merged to for Timber Queensland Ltd where Colin held the position of Manager – Timber Application & Use until his retirement from full time employment in July 2014. For the past 39 years he has been active in all facets of timber engineering and design. His areas of special interest include domestic engineering including high wind design, durability design, stress grading, the design and application of timber in commercial construction, fire resistance design and the development of Australian Standards and Building Codes for timber. Colin has been an active member of a number of Australian Standards Committees including the Timber Structures, Structural Timbers, Timber Framing Code and Timber Preservation Committees for many decades and is still a member of TM – 010 Timbers Structures and Framing. As well as the SAI committees, Colin has also represented Australia on the ISO Committee TC 165 – Structural Timber on numerous occasions.

Colin has been instrumental in the writing and publishing of a number of documents for TRADAC, Timber Queensland, the National Association of Forest Industries, the National Timber Development Council and Forest and Wood Products Australia. These include the (TRADAC) Queensland Timber Framing Manuals, MRTFC publications, AS 1684, some of the Datafiles in the NAFI Timber Manual and the Timber Service Life Design Guide.

Building industry training and education are also a focus of Colin's activities and these have been recognised with achievement awards from Rotary and other community organisations.

Some activities recently completed include the project management for a $6.0+ million National Durability Design Project, R&D into timber framing systems, recycled timber grading rules and timber flooring performance. Other recent activities have included preparation of Design Guides for Forest and Wood Products Australia covering moisture affected timber and lightweight timber systems for noise transport corridors.

Some career achievements include:-

- Development, adoption and implementation of the TRADAC Timber Framing Manual series for high wind house construction throughout Queensland. These publications were 'called up' by The QLD Building Regulations under Appendix 4 and later the design principles and details were embraced by revisions to AS 1684 in 1999 and became applicable to all of Australia via the NCC – BCA.

- The development of timber Durability Design procedures based upon the principles of performance and reliability. These principles have now been embodied in a primary referenced document 'Construction Timbers in Queensland' under a QLD variation to the BCA and satisfy the BCA's implicit durability performance requirements.
- Delivery of 'face to face' timber engineering and technology training, education and continuing professional development to over 50,000 TAFE/Tertiary students, architects, engineers, building designers, landscape architects, teachers, regulators and builders etc. Some examples of these activities have included participation in:
    - Development and delivery of the Graduate Diploma of Structural Timber Engineering. CIAE – Rockhampton
    - Development and delivery of the Continuing Education Program - Timber Engineering. QUT
    - Advanced Structural Materials – Timber. U of Q
    - Engineers Australia – Structural Branch Lectures
    - BSA/QBCC Super Show series
    - AIBS CPD training
    - AIA CPD training
    - BDAQ CPD training
    - QMBA Roadshows
    - TRADAC/Timber Queensland Roadshows and Seminars
- Post high wind and flood damage event assessments to ensure published design and remediation advice is consistent with contemporary recommendations
- Valued/accepted as an independent expert witness in Tribunal and Court deliberations

Awards

Stanley A Clarke Medal – 1999 – Institute of Wood Science Australia for services to the Australian timber industry in technical market development and support.

R W Chapman Medal – 2009 – Institution of Engineers Australia for services to the engineering profession.

1. **Principal Author of the following Publications:-**

- Queensland Timber Framing Manual W33N-W41N - 1994
  Queensland Timber Framing Manual W41C - 1992
  Queensland Timber Framing Manuals W50C - 1994
  Queensland Timber Framing Manuals W60C - 1992

Published by TRADAC Qld.

- Timber Manual
  - Datafile P4 Design for Durability
  - Datafile SP1 Timber Specifications
  - Datafile SS3 Timber Floors – Commercial & Industrial
  - Datafile SS4 Timber Decks – Commercial, Industrial & Marine
  - Datafile SS6 Timber Shearwalls & Diaphragms
  - Datafile P5 Protecting Buildings from Subterranean Termites

Published by National Association of Forest Industries 1989 – 1994.

- Multi-Residential Timber Framed Construction
  - MRTFC 1 Building Code of Australia Fire & Sound Requirements for Class 1, 2 & 3 Buildings
  - MRTFC 3 Structural Engineering Guide for Class 1, 2 & 3 Buildings up to 3 Storeys

Published by National Association of Forest Industries 1996.

- Timber Framed Housing – Design Methodology & Performance Criteria (Limit State Design) MacKenzie C.E. & Juniper P. (Draft)

Published by National Association of Forest Industries 1996.

- Timber Service Life Design Guide 2007.
- Stairs, Handrails and Balustrades 2007, and
- Recycled Timber – Visually Graded Recycled Timber for Structural and Decorative Applications 2008.

Published by Forest and Wood Products Australia

- Timber Queensland Technical Data Sheets Series No's 1 to 33

Published by Timber Queensland Limited 2014

Other publications and papers include:-

- "Resistance of House-Wall Sheeting to Flying Debris, Tech. Paper No. 15. P. Grossman and C. MacKenzie, CSIRO DBR 1977.

- Current Problems and Research – C. MacKenzie, Proceedings AUSTIS Conference, Perth, 1983.

- Design for Engineering Timber Elements and Components, C. MacKenzie, K. Lyngcoln, M. McDowall and J. Pierce, CIAE, 1982.

- Timber Use in Tourist Resorts, C. MacKenzie and P. Law. Southern Engineering Conference, I.E. Aust. 1985.

- Timber in Landscaping, C. MacKenzie, Forum Engineering – Landscape Architecture, The Interface, I.E. Aust. 1985.

- Design for High Wind Areas, C. MacKenzie. Proceedings 22nd Forest Products Research Conference, Melbourne 1986.

- Fire Rated Timber Construction, C. MacKenzie, Proceedings Institute Fire Engineers Conference, Brisbane 1987.

- Innovations in Timber Construction, C. MacKenzie. Proceedings Australian Institute of Building Surveyors, State Conference, Hervey Bay 1987.

- Development of Prescriptive Design Manuals for High Wind Housing in Australia – 1988 International Conference on Timber Engineering – Forest Products Research Society 1988.

- Standardising the Design of Timber Framed Housing, Juniper P. & MacKenzie C. – Proceedings Pacific Timber Engineering Conference – TRADAC 1994.

- Basic Working Loads for Truss Plate Connections in Pinus Elliottii – MacKenzie C. & NcNamara R. – Proceedings Pacific Timber Engineering Conference – TRADAC 1994.

- Development of Fire & Sound Rated Timber Framed Multi-Residential Construction – Dunn A., Collins G. and MacKenzie C. – Proceedings Pacific Timber Engineering Conference 1994.

- Fire Resistance of Timber Framed Floors and Walls – Technical Report 93/5 – Collins G.E.; Collier P.R.C.; and MacKenzie C.E. CSIRO/BTL 1993.

- A Reliability Based Durability Design Method for Timber – An Overview – C. MacKenzie – Proceedings of the 25th Forest Products Research Conference, CSIRO DFFP Melbourne 1996.

- Couran Cove Resort, Environmental Solutions for Ecotourism – C Mackenzie and J Smith. Proceedings 2nd Queensland Environmental Conference. The Institution of Engineers, Australia. May 1998

- Design for Durability – C. MacKenzie – Papers – Timber Structures Seminar, The Institution of Engineers, Australia, Queensland Branch. June 2000.

- The development of prescriptive design manuals for high wind housing in Australia – C MacKenzie – Special Publication – Wind Safety and Performance of Wood Framed Buildings. Forest Products Society, Madison USA Nov. 2000

- Recent Developments in Engineered Durability of Timber Construction. C. Mackenzie, G Foliente, R. Leicester. 7th World Conference on Timber Engineering. University of Mara, Malaysia 2002.

- Regulatory and Consumer Challenges Facing Timber Preservation and Durability Interests in New Zealand and Australia. C MacKenzie. Keynote Paper, International Research Group 34 Conference, Brisbane, Australia 2003.

- Wood Solutions, Timber Service Life Design Guide. FWPA – 2007

- Wood Solutions, Timber Stairs, Handrails and Balustrades Design Guide. FWPA - 2007.

- Interim Industry Standard, Recycled Timber – Visually Graded Recycled Decorative Products. FWPA – 2008

- Interim Industry Standard, Recycled Timber – Visually Graded Recycled Structural Timber Products. FWPA – 2008

- Wood Solutions, Assessment of Moisture Affected Timber Construction Design Guide. FWPA – 2012.

Curriculum Vitae: Lex Somerville

Personal Details
Name: Lex Raymond SOMERVILLE
Address: 29 Norwich Street
Wavell Heights
Brisbane Q 4012
Contact Details: Home: (07) 3350 4970
Work: (07) 3358 1868
Mobile: 0418 737 222
E-mail: lexs@bmccservices.com
Marital Status: Married
Date of Birth: 20 May 1953
Leisure Interests: Aviation
Computers (hardware & software)
2
Academic Qualifications
Brisbane Boys College
- Sub-Senior
Kelvin Grove Technical College
- Senior Geometrical Drawing
Queensland Institute of Technology
- Engineering Drafting
Eagle Farm Technical College
- Apprenticeship - Carpentry & Joinery
Professional Development:
1987 **Queensland Forest Industries Training Council**
- Wood Technology -Timber, The Application 1, 2 & 3
1988 **Queensland Forest Industries Training Council**
- Train the Trainer
1991 **Queensland Forest Industries Training Council**
- Visual Stress Grading – Hardwood
- Visual Stress Grading – Softwood
- Visual Stress Grading – Cypress
Bywater
- Quality Assurance Auditing
1995 TRADAC
- Kiln Drying of Timber
1998 DPI – Forestry
- Timber Species Identification
- Termite and Wood Borer Identification
2002 Hazid Safe
- Construction General Safety Induction
2005 **Marc Ratcliffe Workplace Education and Development**
- Certificate IV in Assment and Workplace Training (BSZ40198)
Personal Development:
1999 Australian Air Flight Training
- Private Pilots Licence ARN 554356
Endorsements: -
- Retractable Undercarriage
- Constant Speed Propeller
- Multi-engine PA 30/39, PA 34

Professional Associations
Member - Queensland Master Builders' Association
Queensland Building Services Authority – House Building Licence - No. 18096

Career History

I have over 40 years of experience in the construction and building materials industry including:
 7 years as a Building Contractor;
 16 years as a Technical Consultant with the Timber Research and Development Council; and
 10 years as an independent building materials and construction consultant.

BUILDING MATERIALS AND CONSTRUCTION CONSULTANT 2003 – PRESENT
Since 2003 I have been self employed as an independent building materials and construction consultant. In this role I strive to provide quality technical advice, testing, inspection and training services to builders, designers, building certifiers, architects, engineers, building associations, government and manufacturers.
I regularly carry out site inspections and written reports relating to building practices, product quality and performance. These inspections range from timber flooring to assessing the stress grade of timber on domestic housing and commercial projects such as the re-development of old commercial timber structures.
Clients include:
 Timber Queensland
 Cyclone Testing Station - James Cook University
 Queensland Building Services Authority
 BlueScope Steel
 National Association of Steel Framed Housing
 Southern African Light Steel Frame Building Association
 as well as various building certifiers.
I am also on staff at James Cook University as casual Research Officer with the Cyclone Testing Station. In this role I was involved with the damage investigations of the 'Gap Storm' in Brisbane in 2008 and Cyclone Yasi in 2010 and a co-author of the reports on these events prepared by the Cyclone Testing Station.

TIMBER RESEARCH AND DEVELOPMENT ADVISORY COUNCIL LTD.(TRADAC) 1987 - 2003
Profile:
The Timber Research and Development Advisory Council Limited (TRADAC) was a research and technical advisory organisation providing professional advice and assistance on timber related issues to timber manufacturers, specifiers, designers and builders.
For 30 years TRADAC operated as a Queensland Statutory Authority (Under the Forestry Act 1959) however from 1 July 2000 TRADAC became a not for-profit company limited by guarantee, membership based organisation.
TRADAC'S mission was to create and maintain a positive market environment for the sale and use of timber and timber products for the benefit of its members.
16/06/2014 4
Position: Technical Consultant 1992 - 2003
Responsibilities:
Generally my role at TRADAC was to provide technical support to the timber and construction industries on the wide range of timber products, construction techniques, and relevant Australian standards. This has been achieved via the phone, personal contact in office or on-site as well as the following activities:
 Technical Publications
I assisted in the development and drafting of TRADAC technical publications such as the TRADAC Technical Data Sheets.
I was also actively involved in the development of the new Timber Framing Code AS 1864 – 1999 as well as other timber industry publications.
 Seminars and Workshops
As well as various TRADAC seminars, I prepare and give presentations regularly at Queensland Master Builders Association (QMBA), Housing Industry Association (HIA), Building Designers Association Queensland (BDAQ) and Institute of Building Surveyors (AIBS) conferences and meetings.
I have also given presentations to Architectural, Engineering and Interior Design students

on the properties and applications of timber.
 Site Inspections
I regularly conducted site inspections and written reports relating to building practices, product quality and performance. These inspections ranged from timber flooring to assessing the stress grade of timber on domestic housing and commercial projects such as the re-development of old commercial timber structures .
 Research Projects
I have been involved directly with various research projects conducted by TRADAC such as the initial ‘Glueing Effect Trials on Timber Flooring’ and the Timber Portal Frame Joint Testing
 MRTFC (Multi-residential Timber Framed Construction)
I provided technical support to Architects, designers and builders in their office and on site for MRTFC (Multi-residential Timber Framed Construction) projects.
 Support for Building Studies Lecturers.
As part of a national timber industry project to assist building studies lecturers keep up to date with timber related issues, I developed and maintained a database of all building studies lecturers in Queensland, Northern Territory and Western Australia. Lecturers are kept informed of the latest in timber products and other timber related issues via a newsletter I write and distribute quarterly.
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I organised and ran annual live-in two-day workshops for building studies lecturers at the Gympie Forestry Training Centre. The success of these is demonstrated by the continuing high demand for this workshop.
As part of this building studies lecturers support program, I developed a training package for lecturers to assist them with the teaching of the Timber Framing Code (AS1684). This package consists of a commentary on AS1684, a video and PowerPoint presentation.
 Construction Industry Training
After the introduction of the revised TRADAC Timber Framing Manuals in 1992, there was a demand for a short training course on the new manuals. I developed a suitable one day course and have regularly presented this course throughout Queensland to architects, designers, building certifiers, builders and timber industry staff. With the introduction of the new Timber Framing Code AS 1864 in 2000, I developed a new short training course on AS1684 and have regularly presented this course throughout Queensland to architects, designers, building certifiers, builders and timber industry staff.
 Timber Industry Training
Apart from training on AS1684, I conducted training courses on Wood Technology, Visual Stress Grading of Hardwood, Softwood and Cypress. During this time I was also involved with the re-writing of the course notes and the general upgrading the courses.
 Home and Trade Exhibitions
I have also been involved with the design, construction and manning of displays at various trade and home shows.
Major achievements in the role are:
 Gaining trust, loyalty and respect from sawmillers, merchants, architects, engineers designers, building certifiers, builders and timber industry staff across Queensland.
 Development of the new Timber Framing Code AS 1864 – 1999

 Development of a training package on the Timber Framing Code (AS1684)
TIMBER RESEARCH AND DEVELOPMENT ADVISORY COUNCIL LTD. (TRADAC) 1987 - 1992
Position: Telephone Advisor
Responsibilities:
My main duty was to answer inquiries from Architects, Engineers, Building Certifiers, Designers, Builders, the general public as well as sawmillers and timber merchants on the correct application and use of timber products.
The inquiries ranged from the TRADAC Timber Framing Manuals to the specification of timber sizes, fixing details, finishing requirements and species selection for a variety of projects from domestic houses, pergolas and decks to large commercial projects such as ‘Expo 88’.
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During this time, a major upgrade of the Framing Manuals was conducted and I was actively involved in the development and writing of the 1991 editions of the TRADAC Timber Framing Manuals.

Major achievements in the role were:

• Initiated the development and co-wrote a book “Building in Timber” aimed to provide the home builder and student with the basic knowledge required for the construction of a conventional timber framed home and to. Guide them through the entire construction procedure from start to finish. “Building in Timber” is still used a reference book by students and owner builders.

A.R. SOMERVILLE & SON 1976 - 1987

Profile:

A.R. Somerville & Son was mainly involved in the construction of domestic dwellings, light to medium industrial buildings and churches.

Since its inception, A.R. Somerville & Son had an excellent reputation for producing high quality workmanship, and for a number of years had the record for building more churches in Brisbane than any other builder.

Position: Proprietor 1982 - 1987

Responsibilities:

In 1982 my father retired and I took over the proprietorship of the business and continued to trade as A.R. Somerville & Son. Apart from the responsibility of running a small business, I continued to design homes and draw plans for clients.

Position: Foreman 1976 - 1982

Responsibilities:

As Foreman I was completely responsible for the construction of a variety of small to medium sized building projects. These included Architect designed homes (one of which won the Architect Design of the Year award) light industrial buildings to 1500 square metres, as well as other factories and domestic dwellings. I was also responsible for liaison with clients, and often designing their homes and drawing the plans.

T.F. WOOLLAM & SON Pty Ltd 1974 - 1976

Position: Carpenter / Leading Hand

Responsibilities:

I commenced as a Carpenter and reported to the Foreman. After six months I was appointed to Leading Hand. In this position I was responsible for carpentry work associated with the building of the Richmond Hospital. This included negotiating and liaison with sub-contractors and suppliers for the building site at Richmond and also for other building sites at Hughenden, Cloncurry and Torrens Creek.

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Apart from the Richmond project, the others were of a smaller nature which included a Railway Station and Quarters, extensions/alterations to a Pre-School and Telephone Exchange. This work encompassed liaison with Consulting Engineers on the projects as well as organising and supervising sub-contractors and the purchasing of materials.

A.R. SOMERVILLE & SON 1971 - 1974

Position: Apprentice Carpenter

Responsibilities:

Whilst my main duties were as an apprentice, I also drew most of the plans for dwellings, extensions and industrial buildings and became familiar with Local Government regulations and by-laws pertaining to domestic and industrial buildings.

R.H. ROBINSON Consulting Engineers 1970 - 1971

Position: Cadet Draftsman

Responsibilities:

I commenced as a Cadet Draftsman with this organisation involved in Structural and Civil Engineering Consulting. Reporting to the Chief Engineer, I was responsible for drafting work associated with high rise buildings, bridges, pipelines and pressure and nonpressure vessels. During this time I also obtained experience as a surveyors assistant.

Referees

Colin MacKenzie **Timber Queensland (formerly TRADAC)**
Technical Manager
(07) 3358 7903
Colin@timberqueensland.com.au
Dr David Henderson **James Cook University - Cyclone Testing Station**
(07) 4781-4340
david.henderson@jcu.edu.au
Graeme Stark *James Cook University – Cyclone Testing Station*
Business Development Manager
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Technical Manager
(07) 3420 4968
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Curriculum Vitae: Dr Geoffrey Boughton

Name	Geoffrey Neville Boughton (PhD)
Position	Director, TimberED Services Pty Ltd Adjunct Associate Professor, James Cook University
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Summary

Geoff has been the Director and principal engineer of TimberED Services Pty Ltd since 1999. He is also an Adjunct Associate Professor in the Cyclone Testing Station at James Cook University. Over the past twenty years, Geoff has worked on several Standards Australia and International Standards committtess.

Geoff has consulted to a range of national organisations including the Australian Building Codes Board, Bureau of Meteorology, Cyclone Testing Station, Forest and Wood Products Australia, Geoscience Australia, Engineered Wood Products Association Australasia and Australasian timber producers, Department of Local Government, Emergency Management Australia, the Office of Energy Safety and the Power Poles and Cross Arms Forum.

Between 1981 and 1984, Geoff was a Research Fellow for the Cyclone Testing Station at James Cook University, Townsville, North Queensland and completed his PhD in 1989. His research focused on the evaluation of the structural actions of housing under wind loads, and included the performance of tests on individual structural components used in houses, the construction of a simulated cyclonic load test rig for use on complete houses and the use of that rig to test four full-scale houses.

Over the past thirty years, Geoff has participated in a number of research projects on the vulnerability of buildings under wind loads for both government and commercial organisations. He has also participated in, and in some case lead a specialist team from the CTS in numerous damage investigations following cyclones and other severe wind events. The findings from these investigations have been incorporated into reconstruction guidelines, and made available to Standards Australia and local government authorities for revision of current standards and building practices.

Geoff was a senior lecturer in structural engineering at Curtin University of Technology between 1986 and 1995. He taught final and penultimate year students in subjects related to the design of timber, steel and concrete structures and also subjects which specialise in design for tropical cyclones and earthquakes. Much of his research activity was directed towards wind engineering problems and the design of timber structures. It included work on low cost housing, the fatigue behaviour of roofing under wind loads, the grading of timber and deterioration of power poles.

In 1991, Geoff was a visiting professor at at the Boundary Layer Wind Tunnel at the University of Western Ontario, Canada and at the University of British Columbia working on the grading of sawn lumber.

Geoff's experience and expertise in structural engineering, particularly in the areas of timber design, manufacture and use; performance of buildings under wind loads; and his skills as an educator make him a highly sought after speaker for national and international conferences, seminars and workshops. He is the principal author of Standards Australia Handbook 108 – The Timber Design Handbook.

Degrees:

- Doctor of Philosophy, James Cook University of North Queensland, 1989
- Master of Engineering Science, University of Western Australia, 1981
- Bachelor of Engineering (Civil, First class honours), University of W.A., 1975

Professional Membership:

- Fellow, Institution of Engineers, Australia
- Member, Structures Panel IE (Aust), WA Division
- Foundation Member, Australian Wind Engineering Society

Committee Membership:

- Standards Australia, member Technical Committee TM/10 Design of Timber Structures,
- Standards Australia, Chair, Technical Committee BD/99 Wind loads on housing
- Standards Australia, member Technical Committee BD/14 Metal cladding
- Standards Australia, member Sub-committee BD/6.2 Wind loading
- International Standards Organisation, Australian Representative, Technical Committee ISO/TC165 Timber Structures,

List of Publications:

138 publications including the following most recent and relevant:

BOUGHTON G. N., FALCK D. J., GINGER J., HENDERSON D. J. and SATHEESKUMAR N. (2014) “Reliability study for performance of timber roof connections under wind forces”, Proceedings World Conference on Timber Engineering, August 2014, Quebec, Canada .

BOUGHTON G. N. and CREWS K. I. (2013) "Timber Design Handbook" 2nd Edition HB108-2013 Standards Australia, Sydney NSW ISBN 0 7337 2057 9, 556pp

BOUGHTON, G.N. (2012) ‘A new era of commercial stress-grading AS/NZS 1748’ Proceedings World Conference on Timber Engineering, July 2012, Auckland NZ .

BOUGHTON, G.N.and HENDERSON, D.J (2012) ‘TC Yasi – Load testing of timber houses on a large scale’ Procedings World Conference on Timber Engineering, July 2012 Auckland NZ.

BOUGHTON, G.N., HENDERSON, D.J., LEITCH, C.J. and GINGER, J.D. (2012) ‘Structural lessons for housing from TC Yasi February 2011’ Procedings Australasian Conference on Structural Engineering, Perth WA June 2012, I.E.Aust.

BOUGHTON, G.N. (2012) ‘Timber – Design, detailing and durability’ Seminar series presentations to Australian Institute of Architects meetings, 9 major centres March 2012, AIA.

BOUGHTON, G.N. (2011) ‘AS1720.1:2010 – Changes’ Seminar presentations to IEAust meetings, Sydney and Melbourne, IEAust.

BOUGHTON, G.N., HENDERSON, D.J., GINGER, J.D., HOLMES, J.D., WALKER, G.R., LEITCH, C.J., SOMERVILLE, L.R., FRYE, U., JAYASINGHE, N.C. and KIM, P.Y. (2011) ‘Tropical Cyclone Yasi Structural damage to buildings’, Cyclone Testing Station TR57.

BOUGHTON, G.N and FALCK, D.J., (2008). ‘Shoalwater and Roleystone WA tornados – Wind damage to Buildings.’ Cyclone Testing Station TR54.

BOUGHTON, G.N, (2007). ‘Tropical Cyclone George – Wind penetration inland.’ Cyclone Testing Station TR53.

BOUGHTON, G.N and FALCK, D.J., (2007). ‘Tropical Cyclone George – Damage to Buildings in the Port Hedland area.’ Cyclone Testing Station TR52.

BOUGHTON, G.N. and FALCK, D.J, (2007) ‘Wind and Building Damage Issues for Coastal Cities’ proc Conf on Planning, Response, Business Continuity, Coastal Cities and Natural Disasters, Pub Communique Australia, Sydney February, 2007.

BOUGHTON, G.N and FALCK, D.J., (2006), ‘Defining statistical methods for monitoring structural properties of machine graded pine.’ Forest and Wood Products Research and Development Corporation.

HENDERSON, D, GINGER J, LEITCH, C, BOUGHTON, G and FALCK, D, (2006), ‘Tropical Cyclone Larry – damage to buildings in Innisfail area.’ Cyclone Testing Station TR51.

BOOTLE K R (2005) “Wood in Australia” rewrite of Chapters 3 and 8, for Timber Development Association NSW

BOUGHTON, G.N and FALCK, D.J., (2003) ‘Learning from Experience in Tropical Cyclones.’ Paper presented at Australian Tropical Cyclone Coastal Impacts Program, Canberra.

BOUGHTON G N (1999) “Tropical Cyclone Vance – Damage to Buildings in Exmouth” Department of Local Government, Perth WA Australia, ISBN 0 7309 2022 4, 88pp.